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163

INDIAN JOURNAL OF POWER & RIVER VALLEY DEVELOPMENT

VOL. XI NO. 11 NOVEMBER, 1961 RUPEES TWO

APR 1 2 1962

MARGARITZEN RESERVOIR. MOLL DAM ON LEFT, MARGARITZEN DAM ON RIGHT—AUSTRIA.

GHT

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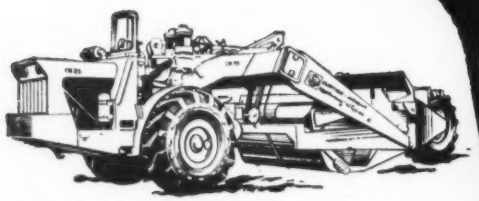
be moved ... to

RIGHT CW-215,
42,000 lbs., a
high range speed of 31.2 miles per hour in fifth gear and a ground clearance of 24" under the scraper, is the rugged answer to all your scraping and disposing problems.



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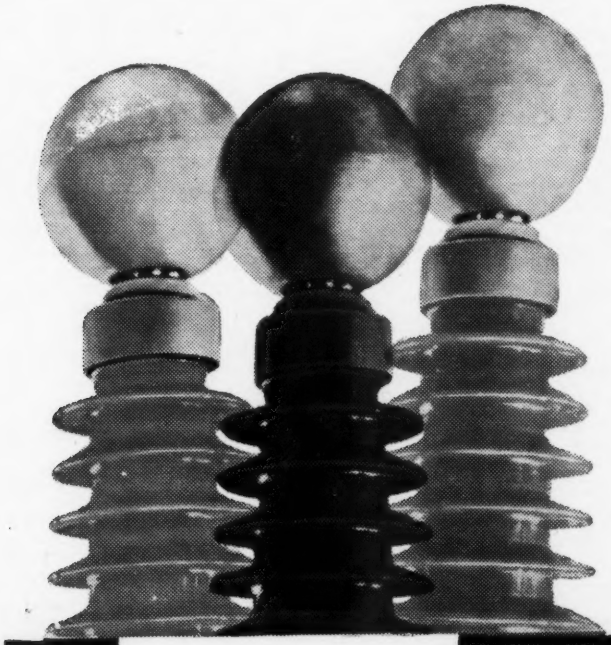

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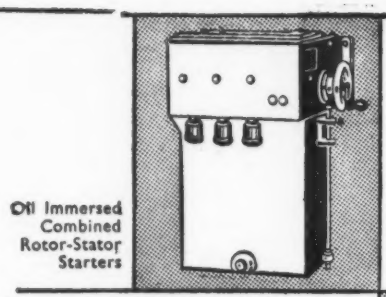
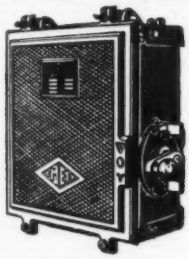
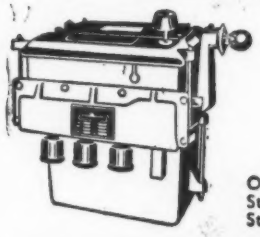


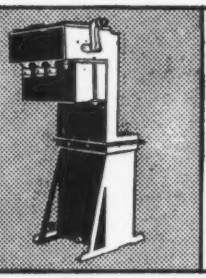
Oil Immersed Combined Rotor-Stator Starters



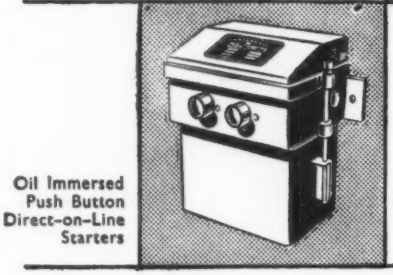
Air Break Star/Delta Starters



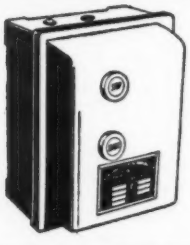
Oil Immersed Star/Delta Starters



Auto Transformer Starters



Oil Immersed Push Button Direct-on-Line Starters



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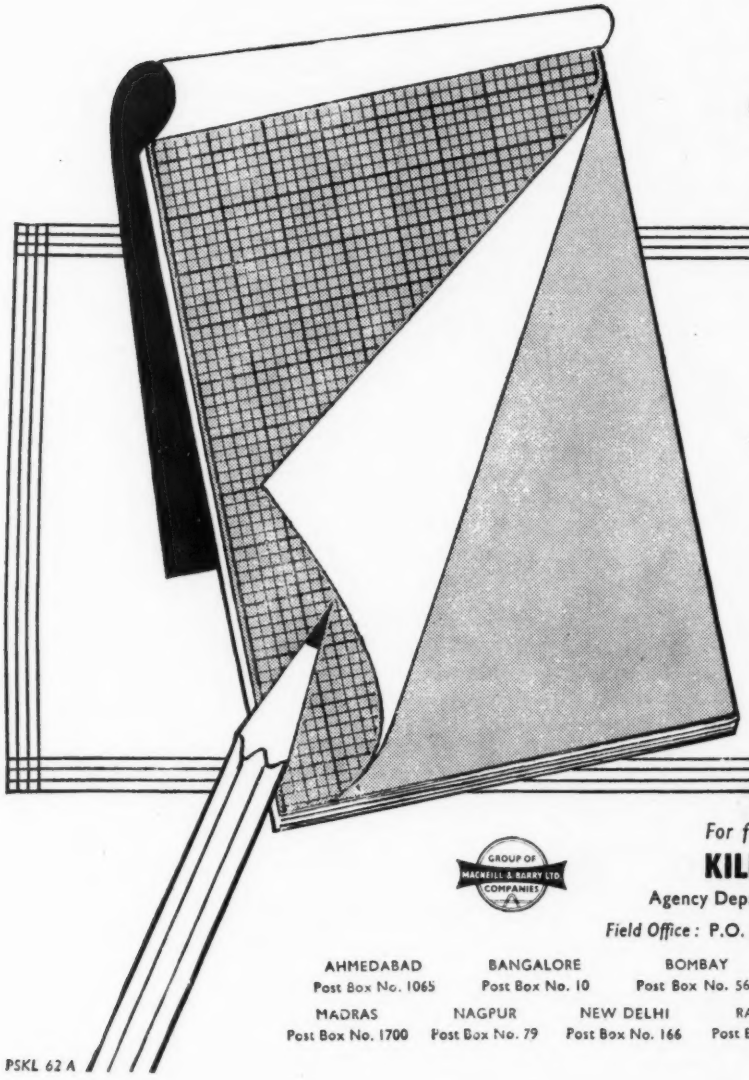

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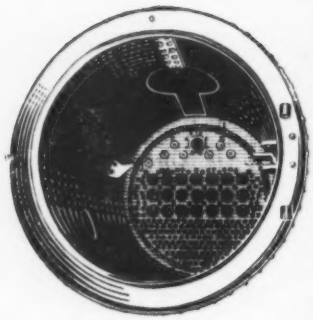

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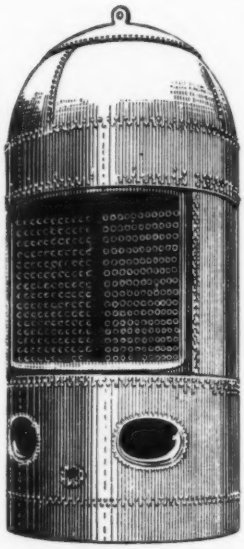

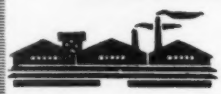

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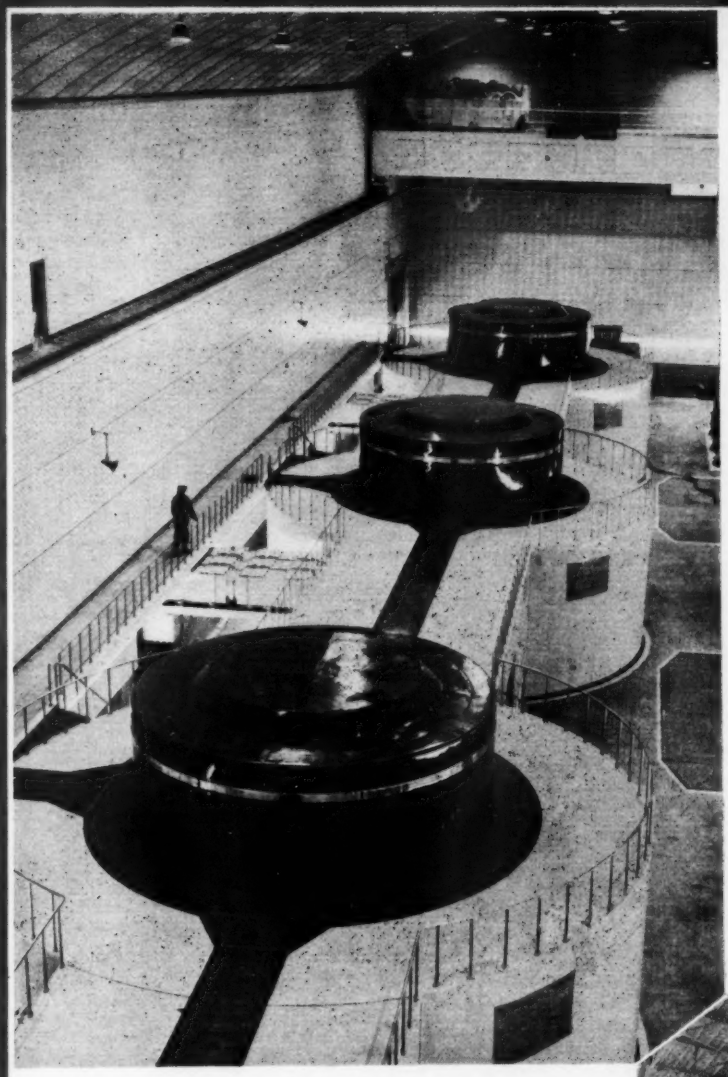

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The set of three generators above—each generating 133,000 kVA—is the second largest of its type in the world.

It has been installed in Japan's newest power development project, the Okutadami Power Station.

The mammoth power transformers with a capacity of 300,000 kVA are also destined to boost prosperity for homes and industries.

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In the manufacture of conveyor belting, experience is a vital factor, for only sound design will ensure quick, efficient and economical flow of materials. The Dunlop organisation has the cumulative experience gained from many different countries of the world where conveyor belting is manufactured.

You can depend on Dunlop to deliver the goods!

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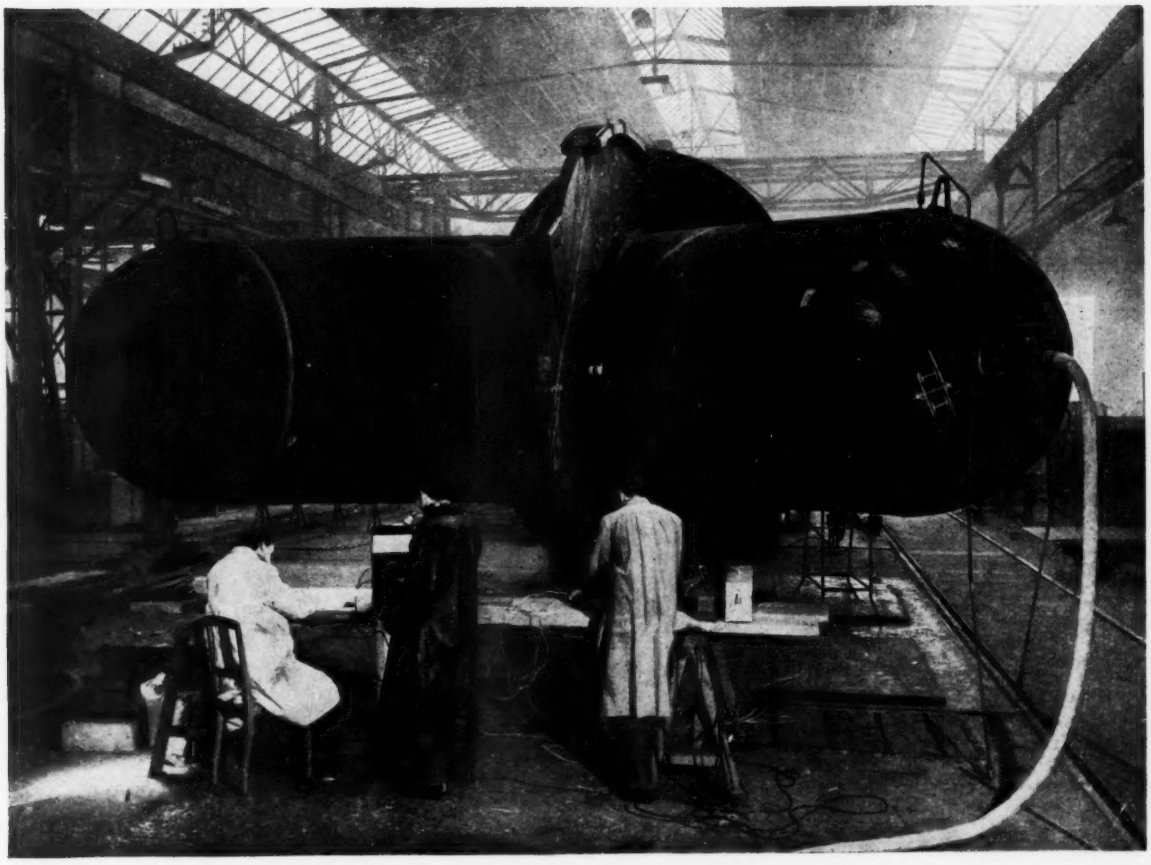

Pressure test and Electric Strain Measuring carried out on the manifold of the KOYNA POWER STATION (Maharashtra State). Test pressure 1420 psi. Diameters 8'6"/2×6'. Thickness 1" 27/32 and 1" 37/64. Weight 40·5 tons.

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for finest performance

Of all aspects of industrial endeavour the most intriguing is thermo-dynamics, for it is this medium that provides industry with its greatest power—steam. Therefore, wherever high pressure steam-raising plant is used, there is a story...a story of experience, analysis and design.

Varying Conditions

Wherever new steam-raising plant is to be installed, either for generating power or in industry, it is essential that skilled investigation and analysis be carried out to ensure that the plant is *tailor-made* to give optimum performance under varying *local* conditions and requirements.

Fuel

Having established the purpose for which the steam power created will be required, it is necessary for the design engineer to identify the type of fuel to be used, either oil, gas or coal. Coal is by far the most widely used fuel, but its thermal content varies and it is therefore necessary to make an analysis to establish the percentage of principal heat-producing elements present. The thermal content of fuel has an important bearing on the designing of the kind of plant to be installed. Generally low-grade coal is used with anything up to 50% ash content. It should be remembered that steam must be raised as economically as possible.

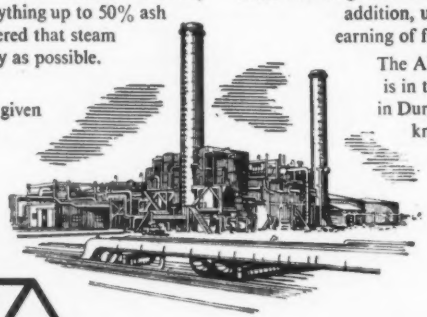
Waste fuel

Particular attention has to be given to industries that wish to use waste material as fuel. For instance, the sugar industry absorbs vast quantities of waste material in the form of bagasse for fuel; again in the pulp and paper industry it is now the practice to combine the processes of chemical recovery and steam raising by burning the 'black liquor' from the digestors in a special type of boiler unit. Of course, these industries cannot keep their steam-raising plant functioning entirely on waste materials. So, in their designing and planning, the technicians and design engineers must allow for varying fuels to be used and at the same time achieve optimum performance.

Powerful Ally

The new and powerful industrial group of ACC-Vickers-Babcock Ltd., with its vast resources of technical knowledge and the rich, wide experience of its partners, will undoubtedly prove a powerful ally in achieving the high target set by the government for the installation of boiler and thermal power plant during the third Five-Year Plan. AVB will be able to draw on the finest engineering minds to study, design and draw up plans that will *tailor-make* not only vital steam-raising plant but also heavy mining and cement-making machinery which will help meet the exigencies of industrial expansion and, in addition, ultimately contribute towards the earning of foreign exchange through export.

The AVB industrial plant at Durgapur is in the final stages of erection. Here, in Durgapur, all the advanced technical knowledge of AVB will be utilised to the fullest extent to ensure that each project is designed for the finest possible performance.



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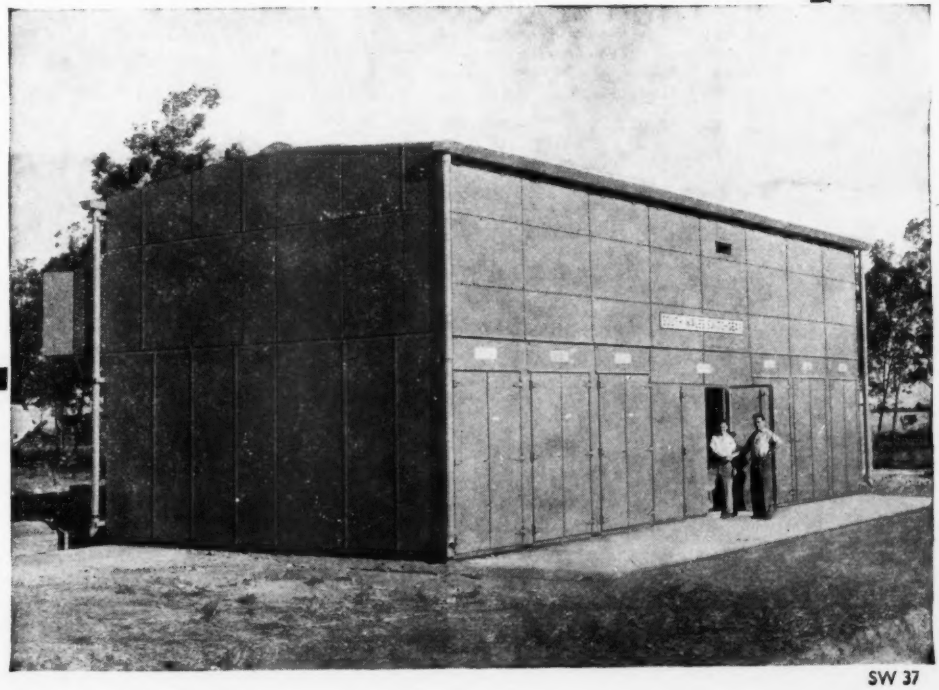

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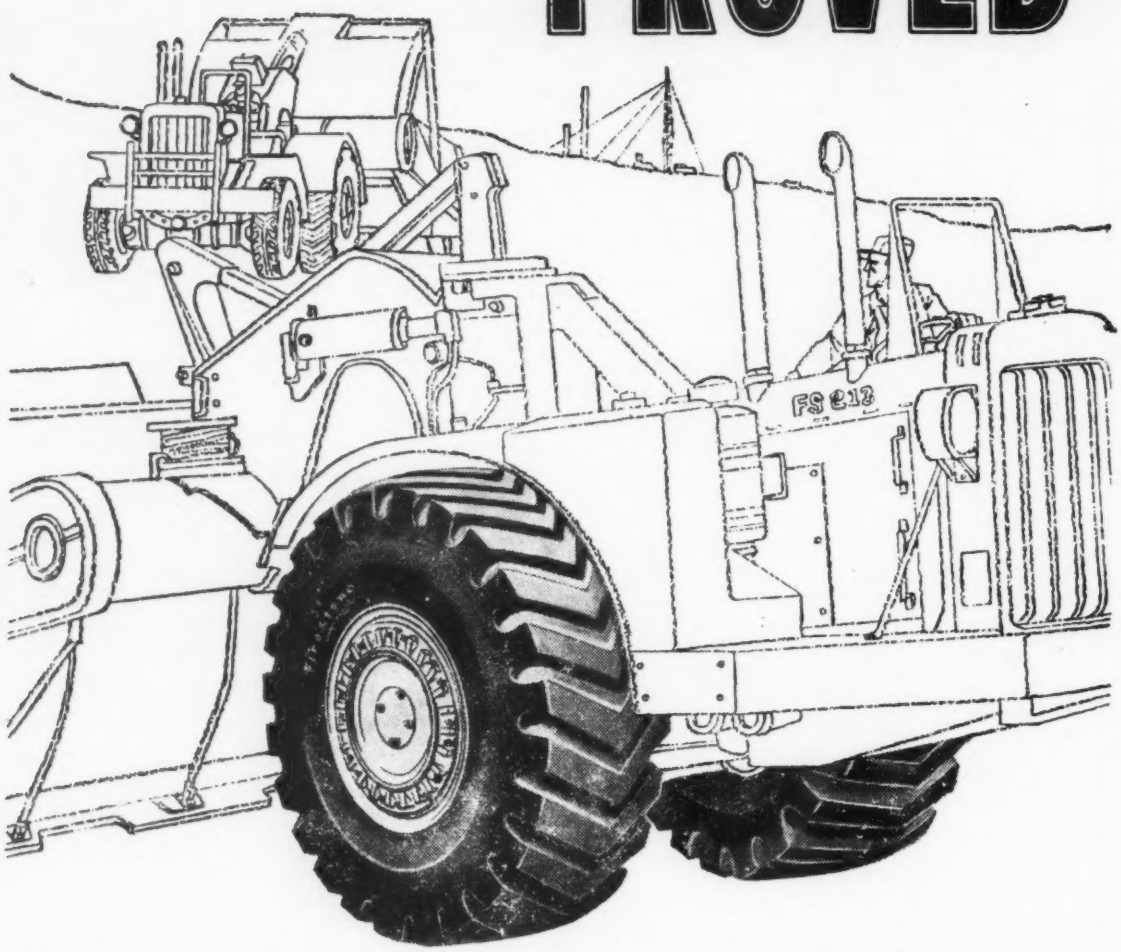

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★ The Firestone Rock Grip Excavator tyre illustrated here has been specially designed for extra pulling power in any service.

★ ADDITIONALLY, the Firestone factory-retreading programme enables you to have worn-down Off-the-Highway tyres retreaded by Firestone in Bombay at a fraction of the original tyre cost. Retread—and cut costs.

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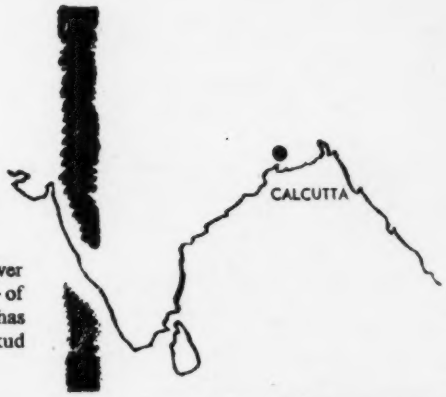
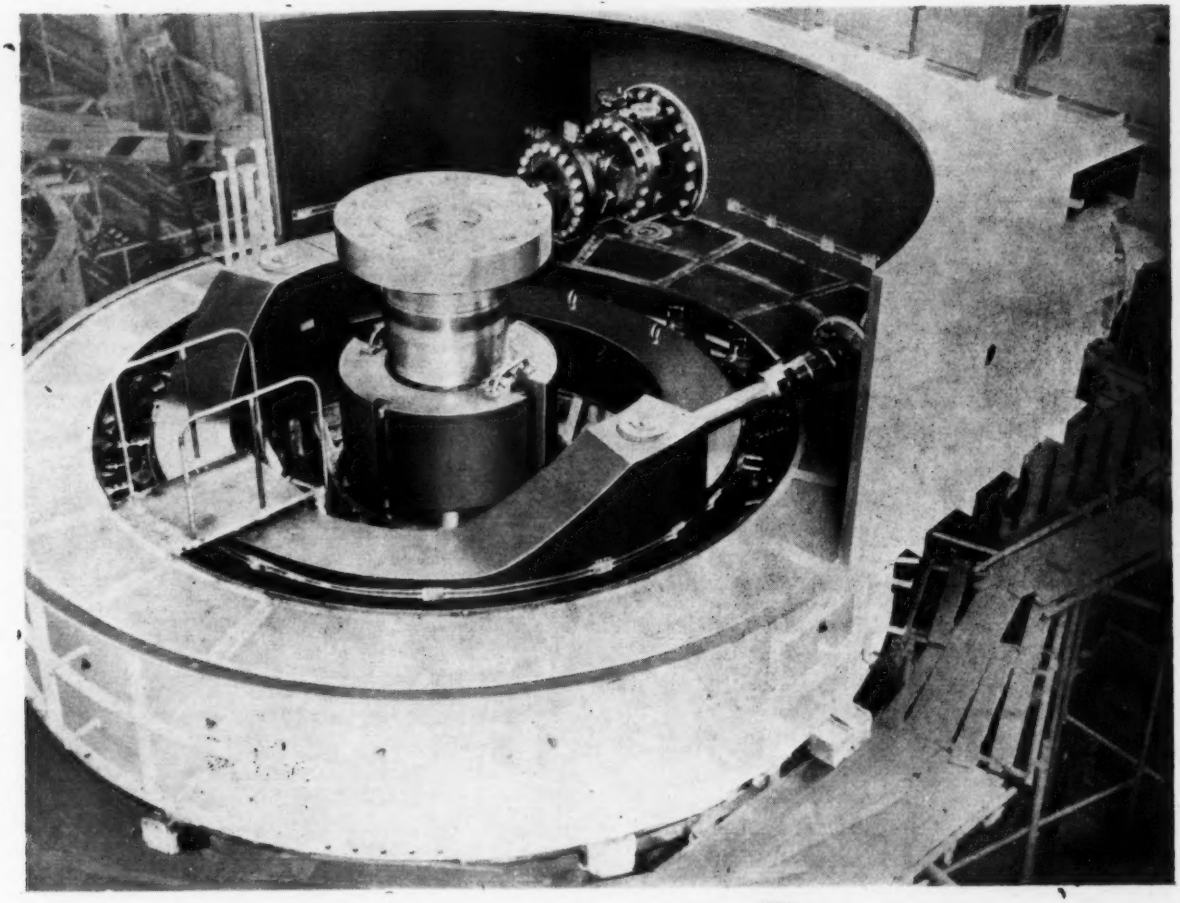


Closely following the official opening of the Bhakra Power Station — equipped with five Francis turbines by Hitachi — of the Punjab State Government, Hitachi, Ltd. of Japan has completed a 52,000 HP Kaplan turbine for India's Hirakud No. 1 Power Station.



Transportation from Japan to India and installation of the new Hitachi turbine — the Hirakud Power Station is 300 miles west of Calcutta — is on schedule.

Hitachi's engineers designed this giant Kaplan turbine for a maximum head of 119 feet, discharge of 5,900 cusecs and a speed of 150 rpm, and its special construction is capable of resisting massive water pressure.

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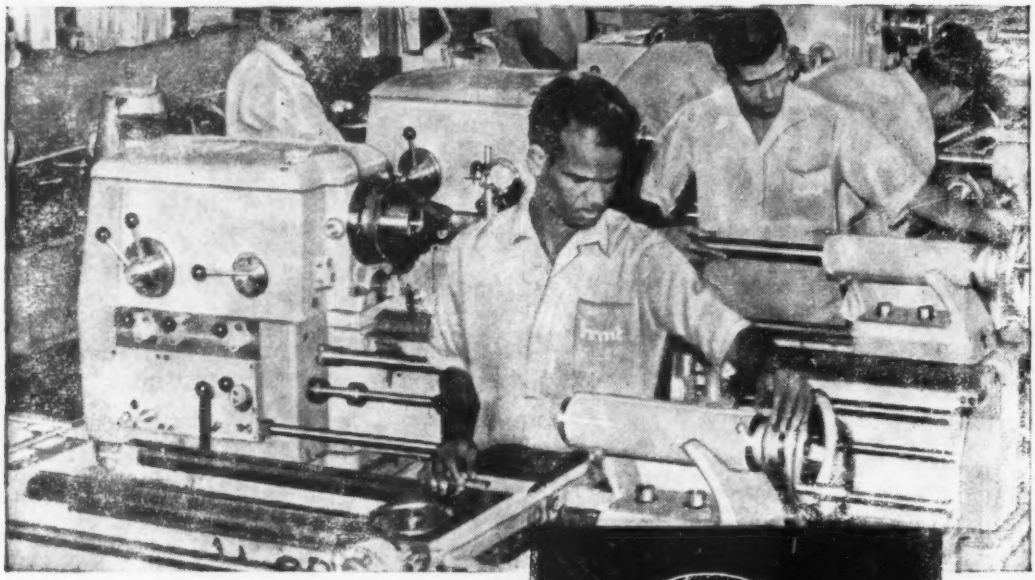

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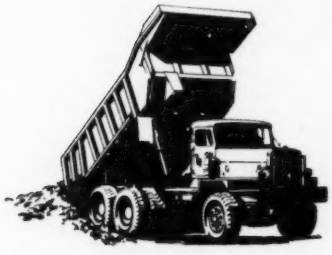

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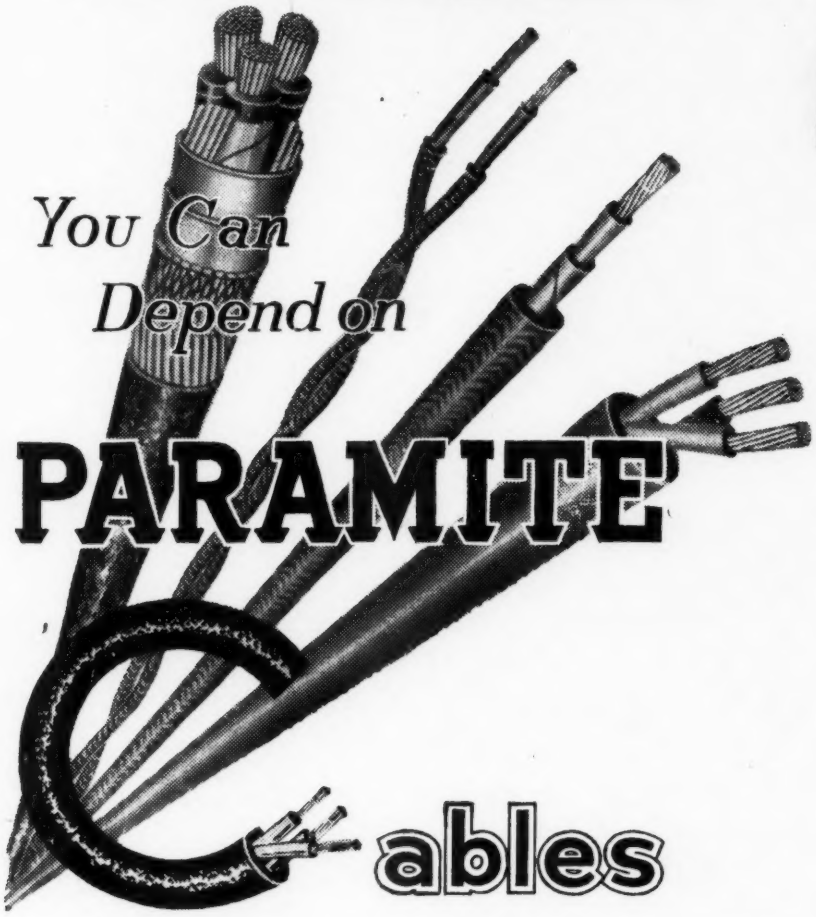

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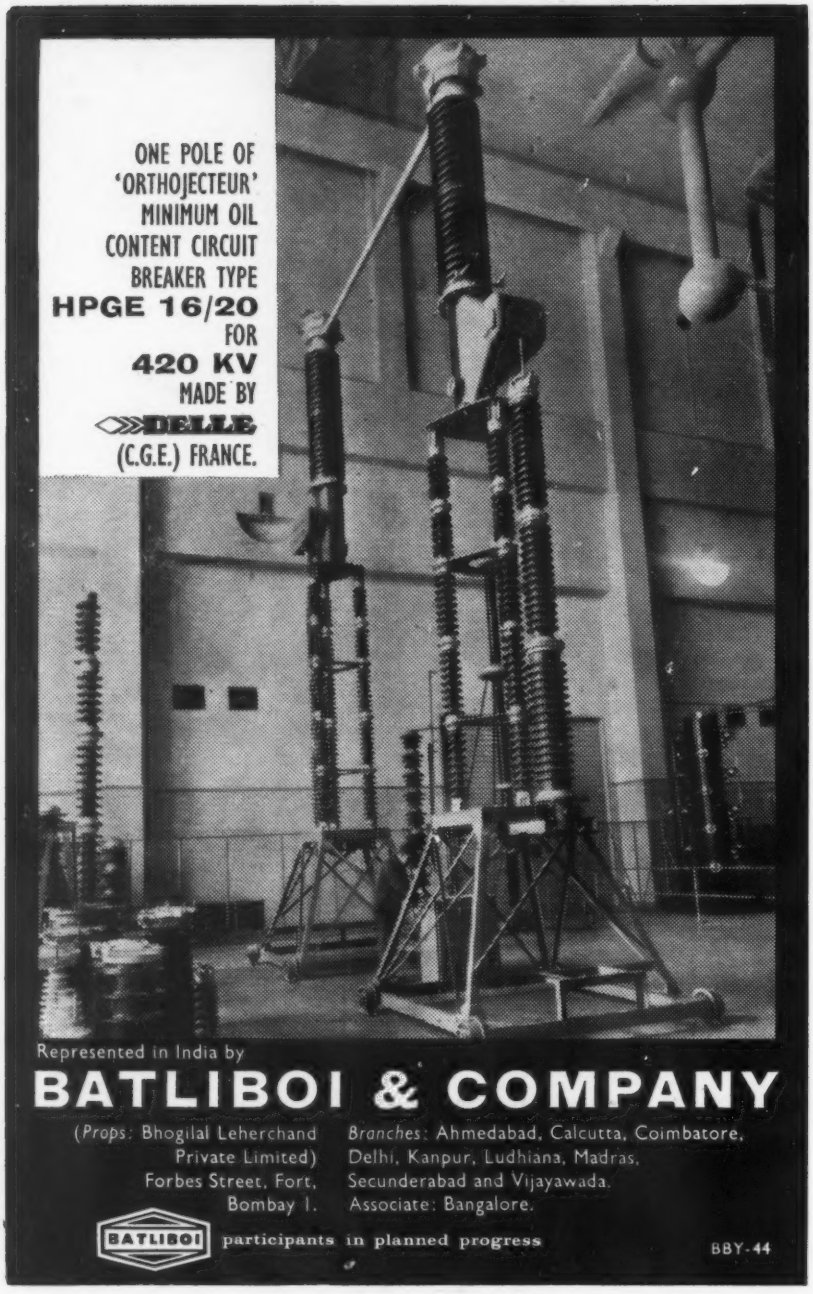
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CONTENTS

Inter-State Water Rights, Water Disputes and Water Laws
by S. R. Vasudev 1

Development of a Tube Well for Anti-Waterlogging
by H. L. Uppal, Gajindar Singh & Amrik Singh ... 4

Power Development in Austria 19

Trends in Hydraulic and Structural Design of Aprons of Barrages on Permeable Soil Foundations—II
by M. V. Rao 22

Homage to Harvey Slocum
Editorial 29

Our Contributors 31

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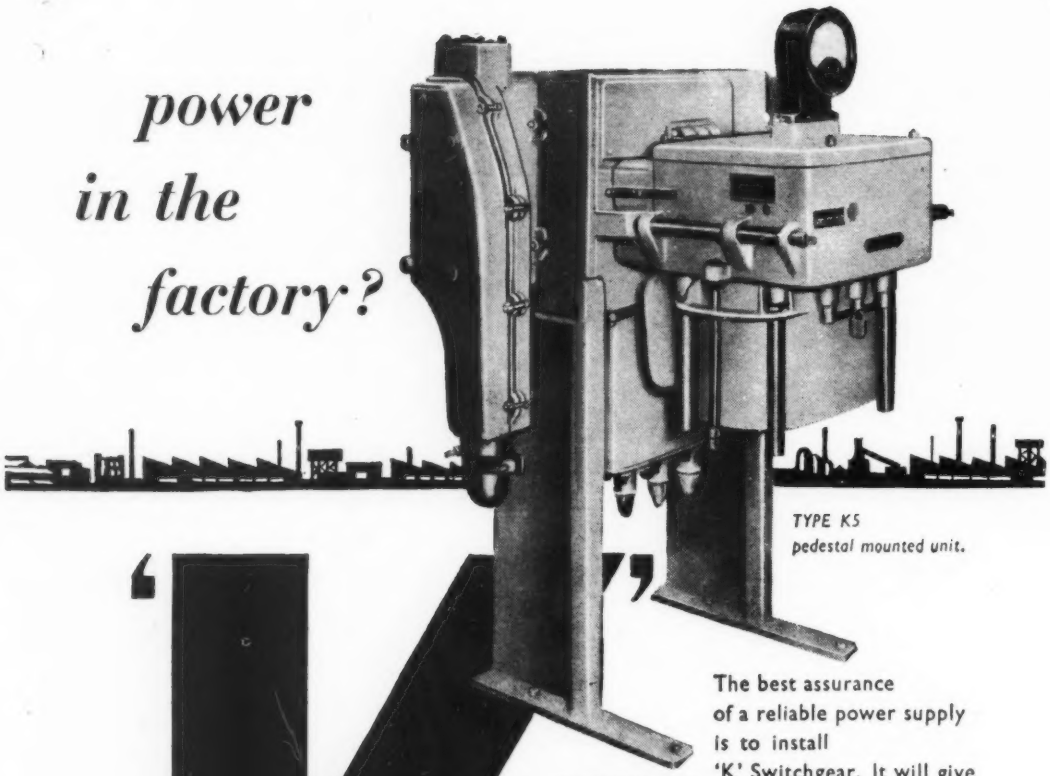

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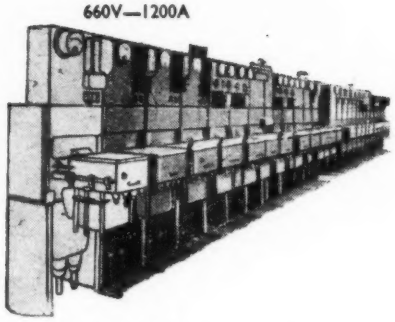

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Inter-State Water Rights, Water Disputes and Water Laws

ALMOST EVERY INTERSTATE RIVER HAS INTERSTATE water problems, big and small, which tend to be of a controversial nature. With the increased demand for water for consumptive use, these problems are becoming more acute leading, at times, to bitter controversies. In recent years, as never before, public attention has been directed towards proper and optimum water utilization. The incidence of droughts, dust storms, soil corrosion, floods, receding water tables, increasing needs for ample and assured water supplies for the expanding industrial areas and the increasing population with consequent larger food requirements and therefore the need for expansion of irrigation, have focussed public attention on the limited water supplies available and their equitable distribution.

When waters of a river have to be shared, disputes are bound to arise. Some of the different factors which might lead to disputes for the use of waters of an interstate stream are :

a. Sharing of water and its utilisation ;
b. Utilization of water by diversion by a riparian state ;
c. The reservoir and canal headworks may be in different states with river forming the boundary ;
d. Nomination of the Agency for the excution of projects wherein the interests of more than one state are involved.

Different Views on Water Rights

Broadly speaking three different views on the subject of rights of states have been advanced from time to time. The first proceeds on what is called the Doctrine of Sovereignity. According to this, every state, by virtue of its sovereignity has the right to do what it likes with the waters within its territorial jurisdiction, regardless of any injury that might result to a neighbouring unit. Pushed to its logical conclusion, this means that a state in which the headwaters of a great river are situated can abstract any quantity of water and make a desert of states lower down. This view is against the trend of international law (the upper state cannot take such a quantity of water from a common river as will seriously impair its utility in the territory of the lower state).

A second view is that the rights of riparian states should be determined by the principle of Common Law which applies to individual riparian owners in England. According to this principle, every riparian proprietor is entitled to the water of a river or stream in its natural flow, without sensible diminution and without sensible alteration in its character or quality. Obviously, this principle would enable a state at the mouth of a river to insist that no state higher up shall make any appreciable diminution in the water which comes down the river. There may be parched areas in the upper state needing irrigation and there may be large quantities of water running waste to the sea past the lower state ; nevertheleess, on this prinoiple of Common Law, the lower state can insist that the waste shall flow down the river without appreciable diminution, even if upper areas may remain dry.

The third view advocates "equitable apportionment", so that every riparian state is entitled to a fair share of the waters of an interstate river. What this fair share is, will depend on the circumstances of each case ; but the river is accepted as for the common benefit of the whole community through whose territories it flows, even though these territories may be divided by political frontiers.

It is obvious that the last mentioned view is the most reasonable and sensible approach and should prevail for resolving interstate disputes over water rights.

Water Rights and Water Disputes

A: WATER RIGHTS:

Water rights should be examined from two standpoints, (a) welfare of the individual water user, and (b) the welfare of the public as a whole. For example the individual must be permitted to use water to the limit necessary for obtaining good crops, but should not be permitted to use it to the extent where the water used by him will yield only a small part of the return which will be received if it is used elsewhere. The water should thus be used on the largest area to produce the largest return.

Each interstate case of water rights should be decided on its own merits. Examination of a case should include questions of the relative necessities of each state as determined by the climatic conditions, environments, natural resources and facilities for utilizing them in the most efficient manner. Due

regard should be given to the fact that nature has placed a limit on this resource, as on others, and that therefore, it is incumbent on all to conserve it and use it in the most efficient manner. A lower state which permits water to flow unused through it, should not be heard to complain of the use of that water by the upper state as unfair apportionment, as this will be against all theories of justice, equity and economics.

Water rights are generally governed by the two doctrines—the Doctrine of prior appropriation and the riparian Doctrine.

The Riparian Doctrine : The riparian doctrine holds that owners of land abutting a stream are entitled to a reasonable use of its water. Riparian rights are proportionate. In times of low flow, each land owner must curtail his consumption or protect the rights of others above and below him. The reasonable use of water by riparian owners, therefore, varies with the available flow. Riparian Doctrine, generally does not encourage water resources development and conservation. The riparian is not obliged to conserve his share of water or use it beneficially. His rights cannot be lost by non-use. The needs of the non-riparians are not taken into account under the riparian doctrine.

The Doctrine of Prior Appropriation : The doctrine of prior appropriation implies first in time is first in service, regardless of property location as long as the use is beneficial. Acrording to this doctrine the acthal use of the water appropriated is essential to the holding of the right. When use ceases, the right is abandoned or forfeited. The priority of use bows to time. If a stream can no longer supply water to all its appreprietors, the most recent users stand to lose. The use to which water is being put in some cases is of relatively minor benefit. The priority system, however, protects such a user as his right has a priority based on time. The first use may not necessarily be the best use.

B : Water Disputes and their Settlement :

It is observed that equitable apportionment is the dominant rule in the decision of interstate river disputes and that in some countries, even as between states recognizing the rule of priority, a prior appropriation has to give way, if it exceeds the equitable share of the state.

a. *General Principles for resolving Water Disputes :* The general principles for distribution and utilization of interstate river waters amongst the states concerned as evidenced from practice in certain countries are :—

(i) The most satisfactory settlement of disputes in regard to distribution of interstate river waters is by agreement, the parties to the dispute adopting the same technical solution of each problem as if they were a single community undivided by political or administrative frontiers.

(ii) If once there is such an agreement that itself furnishes the "law" governing the rights of parties until a new agreement is signed.

(iii) If there is no such agreement, the rights of the several states must be determined by applying the rule of "equitable apportionment", each unit getting a fair share of the waters of the common river.

(iv) The waters of a river should be utilized to the best possible advantage in the tracts commanded by the river, irrespective of territorial boundaries.

(v) It is both just and expedient that each dispute should be dealt with on its own merits. The principle on which the decision is based in one case need not necessarily be applicable to another case.

(vi) In the general interests of the entire community inhabiting dry, arid territories, priority may usually have to be given to an earlier irrigation project over a later one. Priority of appropriation gives superiority of right.

(vii) For purposes of priority the date of the project is not the date when the survey is commenced, but the date when the project reaches finality and there is a fixed and definite intention to take it up and carry it through.

(viii) As between projects of different kinds for use of water, a suitable order of precedence will be :

(1) use for domestic and sanitary purposes ;

(2) use for irrigation and power ;

(3) use for navigation.

In certain cases navigation may have priority over irrigation. In India, however, the Northern India Canal and Drainage Act (1873) and the Bombay Irrigation Act (1879), provide that no compensation is to be awarded for any damage caused to navigation by any project notified under these Acts. Irrigation has thus precedence over navigation.

b. *Specific Decisions :* The following points emerging from the decisions of the Supreme Court in the cass of river water disputes in the U.S.A. should be of interest :—

(i) "Equality of right" as between States ;

(ii) No state can claim exclusive right to the use of all waters within its boundaries. There must be equitable division or apportionment of the benefits of an interstate river between the states affected.

(iii) Water may be taken from one drainage basin for use in another drainage basin although such a diversion may result in some injury to the basin in which such waters originate.

(iv) The burden of construction of storage reservoirs for conserving the surplus-waters of an interstate river otherwise would run to waste, rests upon the lower state, and an upper state is not required to surrender water which it may need to a lower state when such practice would result in waste of the common supply.

(v) The states have the unquestioned authority to agree upon the division and use of the waters of an interstate river, even if such a division and use may have the effect of disturbing and destroying the

rights of individual appropriators which have already been recognized by the State Laws.

(vi) It is not unusual to impose restrictions upon withdrawals by an upper riparian state in order to ensure a minimum gauge height or a minimum flow at places lower down the river, the upper state being required to release water for these purposes from its impounding reservoirs.

(vii) For the purpose of securing an interstate agreement to prevent the waste of national resources such as river water, the Central Government may properly render such financial and other assistance which it consitutionally can.

(viii) It may sometimes be necessary to grant to the lower states the right of inspection of the upper states' dams, reservoirs and other works like metres and other measuring apparatus, of records of inflow, outflow and diverted flow, etc.

(ix) In interstate river disputes costs are as a matter of practice equally divided between the parties to the dispute.

In India two Acts have been enacted under the Constitution for resolving interstate water disputes—the Interstate Water Disputes Act 1956 and the River Boards Act 1956.

The Interstate Water Disputes Act empowers the Central Government to set up an ad-hoc tribunal as and when necessary, for the adjudication of any dispute or complaint in respect of use, distribution and control of waters of any interstate river or river valley. The decision of the tribunal is final and no appeal lies against it.

Tre River Boards Act, empowers the Central Government to set up a River Board in respect of any interstate river it thinks fit, in consultation with the State Governments concerned in regard to regulation or development of interstate river or river valley within its area of operation. Before giving any advice, the Board will try to secure agreement among the State Governments, both with respect to the measures to be undertaken and with respect to the sharing of financial liability involved in undertaking such measures. The Board will prepare new schemes for the regulation or development of the interstate river or river valley and advise the State Governments concerned to undertake work to give effect to such schemes. The Board will also allocate costs of the schemes among the various State Governments. If any State Government does not accept the advice of the Board or does not undertake works in accordance with such advice, the matter can be referred to arbitration by any of the State Governments concerned. The decision of the arbitrator will be binding.

Rights to Underground Water

Underground waters fall into two categories, (i) those flowing in undefined subterranean channels, and (ii) diffused percolating waters. For (i) the same rules as for surface waters are usually applicable, whereas for (ii) either the English rule of absolute ownership or the American modification of reasonable use might apply. According to certain decisions in English Courts, the owner of land containing underground water, which percolates by undefined channels and flows to the land of a neighbour, has the right to divert or appropriate the percolating water within his own land so as to deprive his neighbour of it. In another case the decision on whether the owner of land had a right to sink a well upon his own premises and thereby abstract the subterranean water percolating through his own land which would otherwise, by gravity, have found its way into springs feeding a certain river, the flow of which had been enjoyed by the complainant over a number of years, was that the land owner had the right to sink the well whatever his purpose might be even if wholly unconnected with the enjoyment of his own estate. In another case it was decided that the owner of a land had the right to sink a well in his own land even if it interferred with the underground water feeding certain springs appropriated for feeding water to a town. In certain parts of India every owner of land has the right to collect and dispose of all water, under the land within his own limits, which does not pass in a defined channel. If, therefore, the owners of land exercise their right to abstract water by sinking wells, the volume of "return flow" percolating back into the river is liable to be reduced.

Conclusion

Disputes often arise over minor matters and in the case of water, which is the mainstay of human existence, disputes are bound to arise. But, given the will to come to an agreement, no dispute will be difficult to resolve. Precedents exist and they are invaluable for leaning on to arrive at an amicable settlement. Forced decisions are never lasting, but it is agreement by consent which endures and it is this latter which should be aimed at.

The opinion expressed by the author are his own and do not necessarily represent those of the United Nations.

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Development of a Tube Well for Anti-Waterlogging

During the past few years waterlogging in the State of Punjab has very much increased and millions of acres of fertile lands have become barren and thur. This called for an immediate action to evolve suitable remedial measures.

Amongst the different measures, use of tube wells was considered to be the quick and direct method of lowering water table. Investigations were, therefore, undertaken to evolve a suitable type of tube well, which while working for irrigation purposes would effectively lower the water table and remove waterlogging.

Experiments were carried out on four tube wells with different lengths of filter varying from 24.0 ft to 80.0 ft and a length of blind pipes 10.0 ft to 12.0 ft below sub-soil water level. Effect of each set on the rate and extent of lowering of water table, zone of influence and ground water divide were studied. Of all these, shallow tube wells with filter of 35.0 ft length and blind pipe of about 15.0 ft. length below sub-soil water table was observed to function most satisfactorily and is therefore recommended for tube wells for anti-waterlogging purposes.

WITH THE ADVENT OF CANAL IRRIGATION, USE of percolation wells had been on the decline and as such withdrawals from the underground reservoir have been decreasing. On the contrary additions due to seepage have been increasing. As a result of this, water table all over the state has been gradually rising. During the past few years, however, this rise has been very rapid at the rate of 10.0″ to 12.0″ a year and has produced waterlogging far and wide in the state.

Whether this has been due to increased irrigation or increased precipitation and inadequate drainage or due to construction of network of roads and railway embankments and subsequent blockade of natural drainage systems or otherwise, millions of acres of the once very fertile and cultivable lands have become barren and *thur*. This has seriously affected the prosperity and economy of the state and has been a cause of great concern to the Government and the people alike and has called for an immediate action.

Amongst the many preventive and remedial measures such as construction of embankments on rivers and torrents, construction of open drains, lining of irrigation channels, construction of subsurface drainage system etc., installation of tube wells which directly draw from the underground reservoir is considered to be one of the most quick and direct method of lowering high water table and removing waterlogging conditions. Existing tube wells generally sunk deep to depths beyond 200 ft were not found suitable for this purpose. These appeared to be drawing water from deeper zones and did not effectively lower the sub-soil water level so quickly. This necessitated immediate investigation of a suitable type of tube well which would draw directly from the surface zones of the underground reservoir and cause quick and effective lowering of water table. Such tube wells would serve both for the irrigation and anti-waterlogging purposes simultaneously.

Statement of the Problem

Tube wells in vogue are generally sunk to 200 ft. to 300 ft. depth or more. These, although constantly withdrawing water from the underground reservoir, have not been found helpful in lowering the water table. It has therefore been desired to evolve a suitable type of tube well, which while functioning for irrigation purposes, would also effectively lower the water table or keep it under control and prove helpful for anti-waterlogging purposes.

Statement of Studies

In this connection the following different studies were made :—

Study 1 : A study of the functioning of shallow tube wells and their effect on lowering sub-soil water table.

Study 2 : A study of the functioning of moderately deep tube wells and its comparison with the working of shallow tube wells.

Study 3: Effect of discharge of tube wells on the rate and extent of lowering water table.

Tube Well Research Station

For the purposes of the above studies experimental tube wells were installed at the Tube Wells Research Station near Octroi Post on Amritsar Batala Road. Before doing so, necessary tests were made to determine the location and direction of flow of water bearing strata formation and quality of ground water.

Sub-Strata Formation and Quality of Ground Water : Four trial bores varying from 50 ft to 100 ft, depth were made. Sub-soil water level was of the order of 4.0 ft. or so. Soil samples were recovered from every 1 ft boring and water samples were collected at every 10 ft. interval. Necessary profile notes and strata charts were prepared and are shown in Table 1 and Fig. 1 respectively. Results of analysis of the soil samples are given in Table II.

From the perusal of these it was observed that

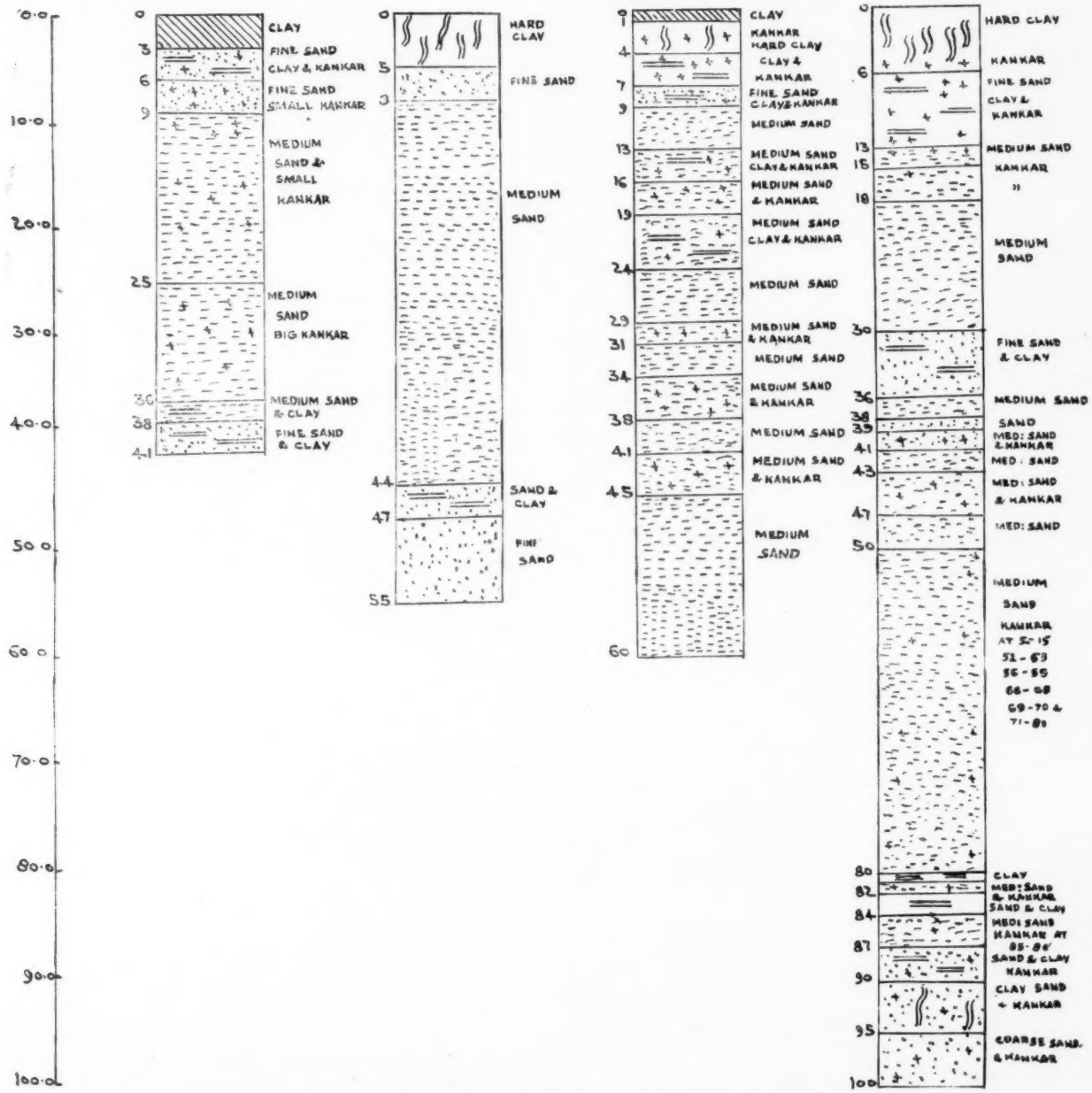

Fig. 1. Development of a suitable type of tube well for anti waterlogging. Strata charts of bore-holes.

clay or clay with kankar occurs in the top crust up to 7.0 ft to 9.0 ft depth and fine or medium sand of an average diameter of 0.22 mm. occurs in the next 65.0 ft. to 70.0 ft. depth. Thereafter coarse sand intercepted by clay bands of 1.0 ft to 6.0 ft thickness occurs. PH value of these soils samples varies from 8.00 to the maximum of 8.90 and total salt contents vary from 0.07% to a maximum of 0.91%. T.S. of 0.91% occurs at a depth of 7.0 ft below natural surface. The quality of water collected from different depths was good and found fit for irrigation purposes.

Installation of Experimental Tube Wells : Four tube wells of 6 inches diameter with different lengths of strainer and blind pipes as given below were sunk.

Tube Well No.	*1*	*2*	*3*	*4*
Length of blind pipe	15.0′	12.0′	15.0′	15.0′
Length of strainer	24.0′	40.0′	40.0′	80.0′

To make these experiments economical, a cheap type of strainer made of iron rod with double coir rope winding was used Fig. 2. This very much

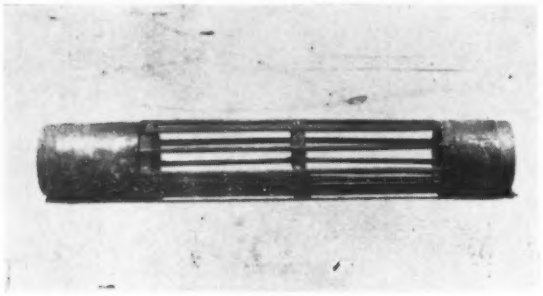

Fig. 2a) A view of the frame work of the strainer.

reduced the initial cost of these experimental tube wells.

A number of piezometers with four feet filters of 100 mesh sieve were fixed at different distances all around for observing the effect of tube wells on lowering of sub-soil water table Fig. 3. A v-notch

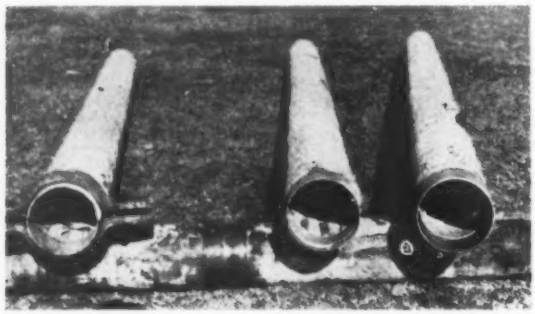

Fig. 2(b) A view of strainer after double coir rope windings.

was fixed below delivery pipe of each pump for measuring discharges.

Operation of Experimental Tube Wells : All the four tube wells were run one after the other with the same set of motor pump and under similar conditions. At every tube well, 5.0 H.P. Kirloskar monoblock motor pump 4.0″ x 4.0″ was used.

STUDY 1

A Study of the Functioning of Shallow Tube Well and its Effect on Sub-Soil Water Table

Experiments were taken up first on a tube well with a filter length of 40.0 ft. and length of blind pipe equal to 15.0 ft. Tests were carried out to examine :—

i. The rate and extent of lowering of sub-soil water table.
ii. Extent of lowering at different distances after different intervals.
iii. The zone of influence and ground water divide.
iv. Effect of discharge on zone of influence and ground water divide.
v. Effect of time on discharge of tube well.

Shallow tube well was run continuously for 50 hours and necessary discharge and water table observations were recorded at different intervals. (Table III). Draw-down curves were prepared and are shown in Fig. 4.

From the perusal of this data it was observed that :—

i. It caused quick lowering of the water table ; within the first five minutes and fifteen minutes sub-soil water table at 5.0 ft. distance from the tube well dropped by 3.20 ft. and 3.95 ft. respectively. Thereafter the rate of fall decreased and water table became almost steady after 8 to 10 hrs (Fig. 5). At other piezometers also the rate of fall occurred in a similar manner, except that it became constant after 12 to 16 hrs.
ii. Effect of tube well on all sides was almost similar and gave similar pattern of draw-down curves.
iii. Maximum drop at 5.0 ft., 25.0 ft. and 90.0 ft distances were of the order of 5.22 ft, 3.12 ft., and 1.33 ft. respectively.
iv. The zone of influence, ground water divide and lowering of water table was different with different discharges (Figs. 6 and 7). The maximum zone of influence with 0.56 cs discharge was 130 ft. and ground water divide was at 495 ft.
v. With 0.3 cs and 0.5 cs discharges, ground water divide was of the order of 300 ft. and 460 ft. respectively (Table IV).
vi. The discharge of tube well did not decrease with time and remained constant. To start with, it gave a slightly higher value but it became steady after 2 hrs.

Conclusions : Shallow tube wells appear to draw

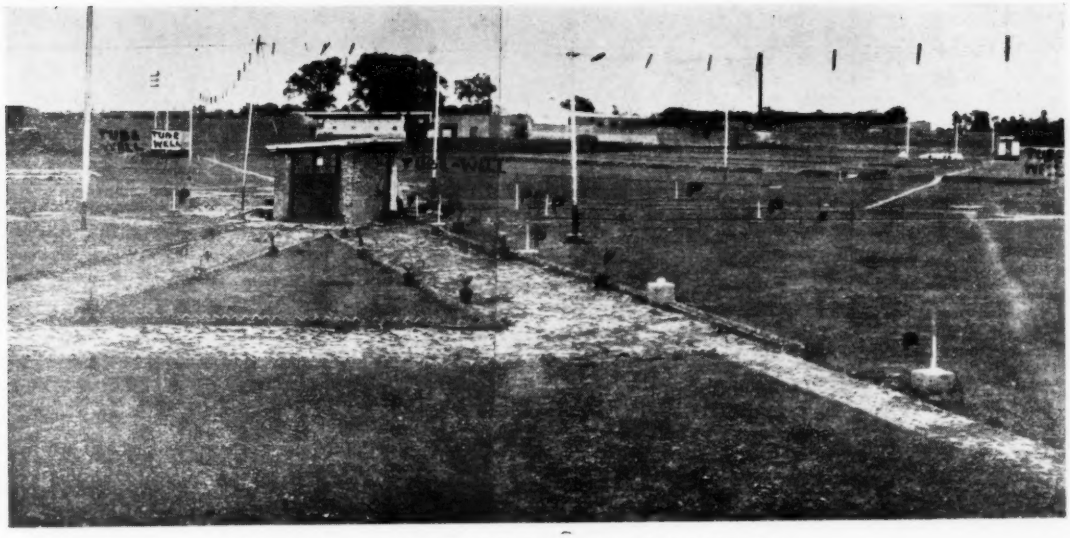

Fig. 3. A view of Piezometers (P) fitted around the Tube Well.

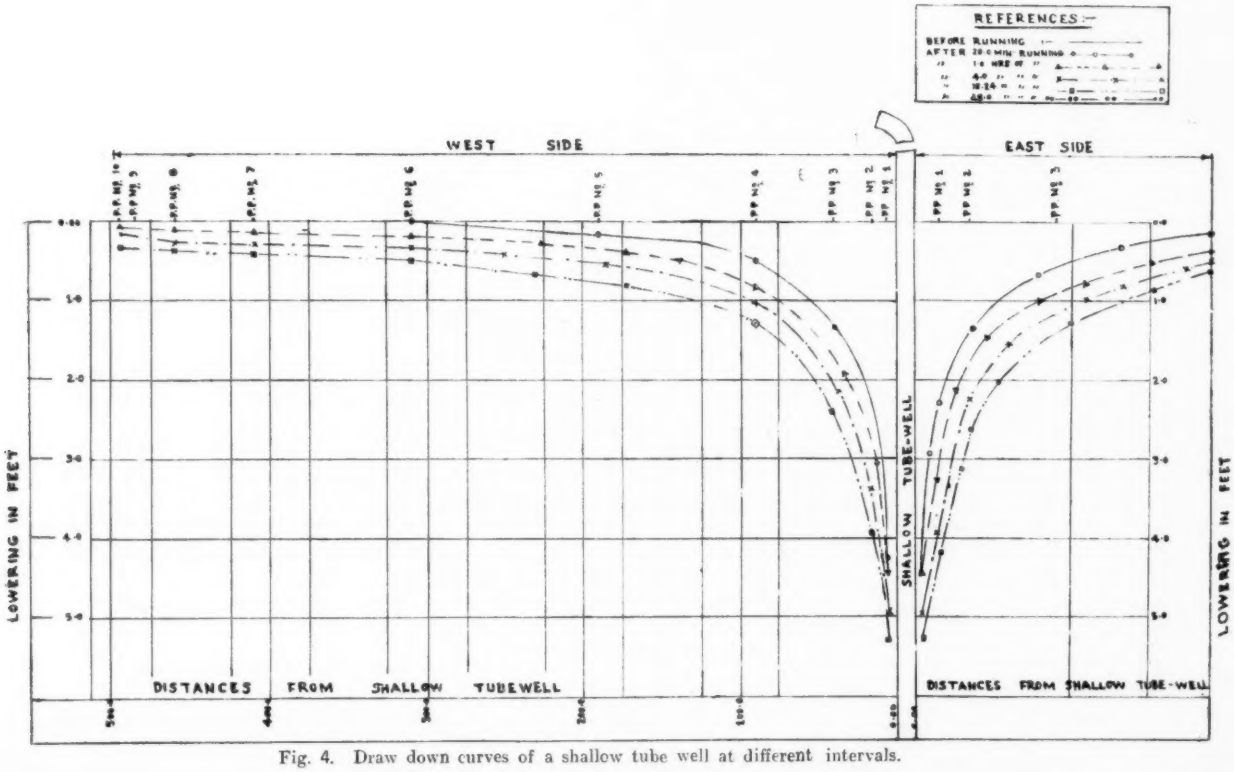

Fig. 4. Draw down curves of a shallow tube well at different intervals.

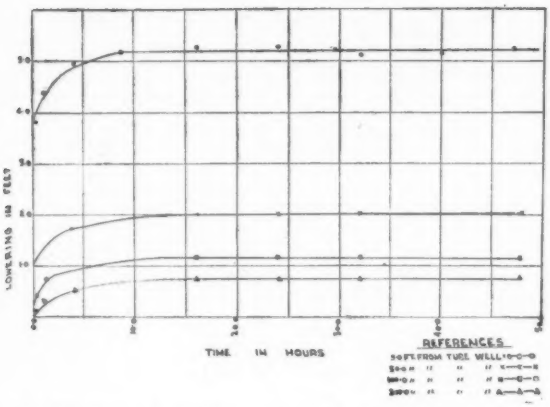

Fig. 5. Drop in sub-soil water level in different Piezometers.

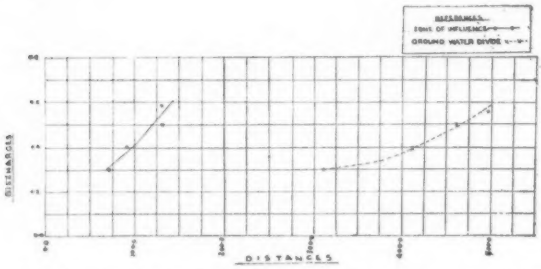

Fig. 6. Showing zone of influence and ground water divide with different discharges.

directly from the surface zones of ground water reservoir and effect quick lowering of water table.

STUDY 2

A Study of the Functioning of Moderately Deep Tube Well and its Comparison with the Working of Shallow Tube Wells

Before sinking a moderately deep tube well at the experimental station, observations were made first on an existing private tube well. This was done just to obtain some preliminary ideas before taking up detailed investigations.

The private tube well had been sunk to a total depth of 100 ft. It had 20 ft length of blind pipe at the top and 80.0 ft length of double coir rope filter below. Its discharge was 0.54 cs. almost the same as that of the experimental shallow tube well.

This was run continuously and sub-soil water level observations were taken through piezometers installed at 5.0 ft, 10.0 ft, 50.0 ft, and 100.0 ft. from the tube well (Table V). Its draw down curves were prepared and are shown together with those of shallow tube well in Fig. 8.

It was observed that after four hours of its working water table at 5.0 ft, 10.0 ft, 50.0 ft, and 100.0 ft, distances had dropped by 0.85 ft, 0.82 ft, 0.62 ft, and 0.40 ft. respectively. Even after 10 hrs, it was just the same. The depth of lowering had been 0.92 ft, 0.87 ft, 0.77 ft, and 0.53 ft, respectively as against 5.16 ft, 4.70 ft, 1.95 ft, and 1.31 ft, in the case of shallow tube well. This indicated clearly that rate and extent of lowering caused by deep tube well were much smaller than that caused by shallow tube well.

In order to confirm these findings and to have correct comparison, it was considered essential to have both these sunk at the same place and in the same strata, so that these could function under similar circumstances. Accordingly, a deep tube well of 15 ft. blind pipe and 80 ft. filter length was also sunk nearby the shallow tube well. 5 B.H.P. Kirloskar motor with 4″ x 4″ pump was fitted on this also.

It was run continuously for over 50 hrs. and discharge and water table observations were taken as before (Table VI). This gave a discharge of 0.76 cs. It caused a maximum lowering of 4.21 ft at 5.0 ft distance from the tube well.

To afford a correct comparison, discharge of this deep tube well was reduced to 0.50 cs—the discharge of shallow tube well and necessary observations were repeated (Table VII). It was seen that maximum lowering of sub-soil water table at 5.0 ft distance come to 2.5 ft. only as against 5.12 ft. with shallow tube well. Comparative statement of lowering of water table and draw down curves in the two cases are shown in Table VIII and Fig. 9 respectively.

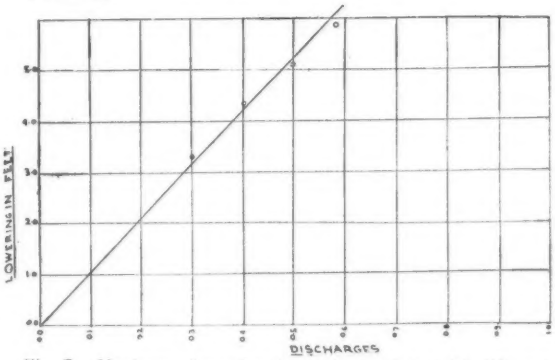

Fig. 7. Maximum drop of water table at 5.0 ft. with different discharges.

From the perusal of the above data it was observed that :—

i. Rate and extent of lowering of water table with deep tube well was much smaller than with the shallow tube well. It was 2.5 ft as against 5.12 ft. at 5.0 ft. distance.

ii. The zone of influence and ground water divide also were much smaller. These confirmed the above findings and amply proved that water table lowering caused by shallow tube wells was quicker and more than that produced by deep tube wells.

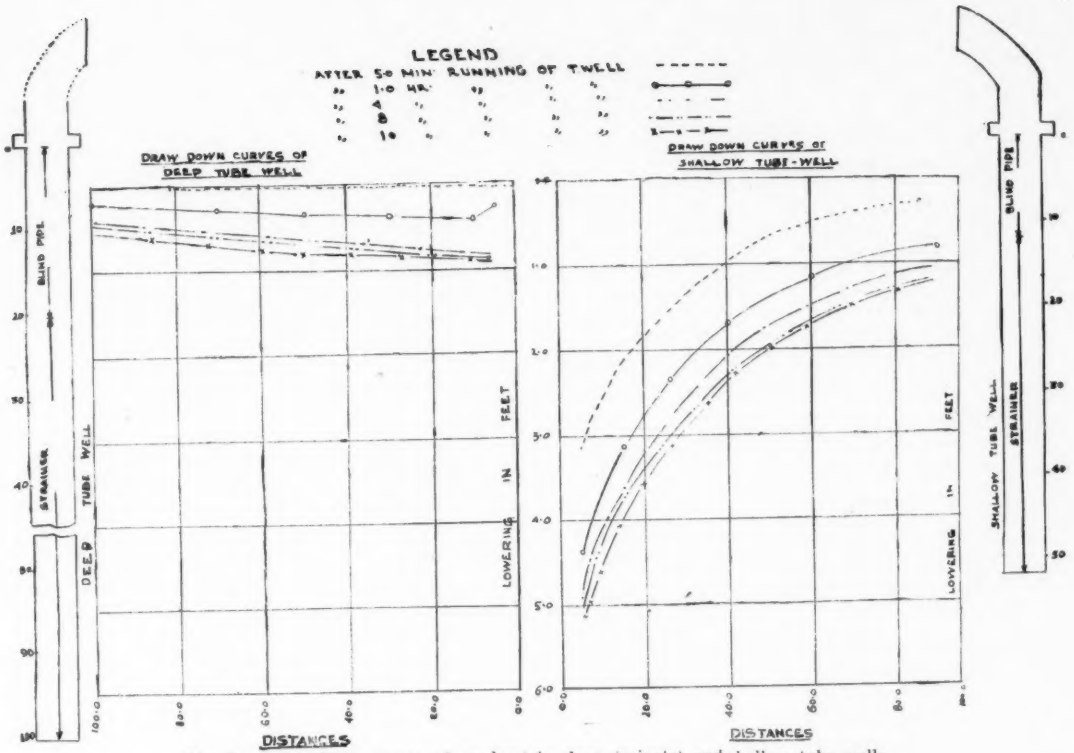

Fig. 8. Draw down curves of moderately deep (private) and shallow tube wells.

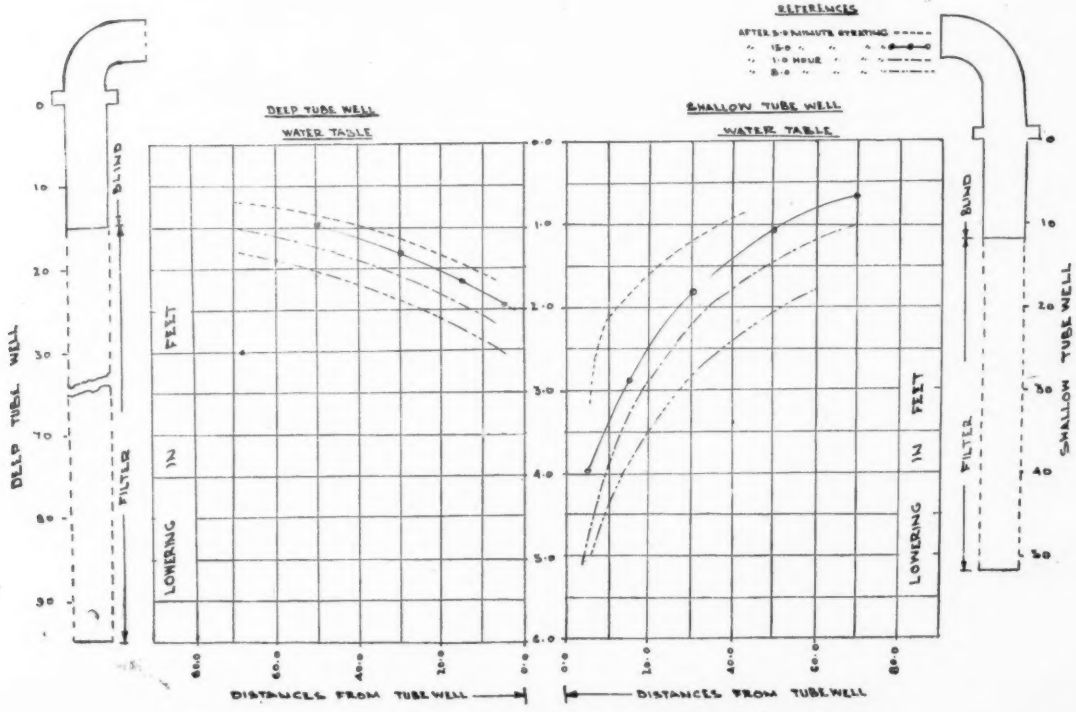

Fig. 9. Comparison of lowering of water table with deep and shallow tube-well.

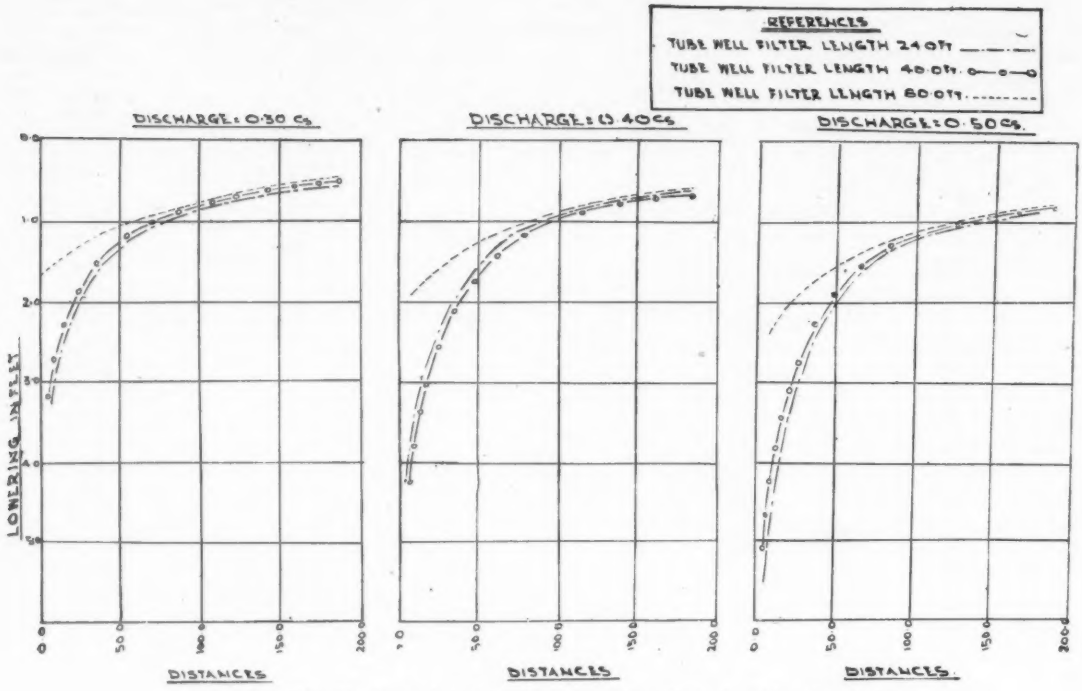

Fig. 10. Maximum lowering of water table obtained with tube-wells.

In order to obtain a still more striking comparison, yet another and still more shallow tube well was sunk at the same site and in the same strata. It had a filter length of 24'-2" only and the length of blind pipe equal to 15 ft.

This and the deep tube well were run with the same motor pump and under similar conditions of discharge etc. and observations were repeated (Table IX). The maximum lowering obtained in the three cases viz with deep tube well, with shallow tube well of filter length equal to 40 ft and with a still shallow tube well of filter length equal to 24 ft. only are shown in Fig. 10.

It was observed that :—

i. Lowering of water table produced by the shallow tube wells was much more than that done by the deep tube well, and as such would be more effective and useful in waterlogged areas.

ii. Lowering obtained with shallow tube well of 40 ft filter length or 24 ft filter length for the same discharge was almost the same.

iii. Shallow tube well of 24.0 ft. filter length could give only a limited discharge of the maximum of 0.496 cs.

Conclusion: Rate and extent of lowering of water table caused by shallow tube wells are much more than that done by the deep tube wells. Tube wells with filter length of 30.0 ft and a length of blind pipe equal to about 12.0 ft. below water table may be adopted. This would be more effective and useful in waterlogged areas.

STUDY 3

A Study of the Effect of Discharge on Rate and Extent of Water Table lowering

In order to examine the effect of discharge on rate and extent of lowering, deep tube well was run for 8 hrs. each for discharges varying from 0.2 cs to 0.7 cs. and necessary observations ware taken. These were repeated on shallow tube wells also. Results of observations are shown in Table X and Fig. 11. From the examination of these results it was observed that :—

i. Greater the discharge, greater was the depth of lowering. This was directly proportional to discharge.

ii. Effect of lowering was pronounced more in closer regions than at distant places.

Conclusion: Increase of discharge of shallow tube well increases depth of lowering. This is more pronouced in the closer regions than at distant places.

SUMMARY OF RESULTS

1. Moderately deep tube wells draw water from deeper zones where as shallow tube wells draw directly from the upper zones of ground water reservoir.

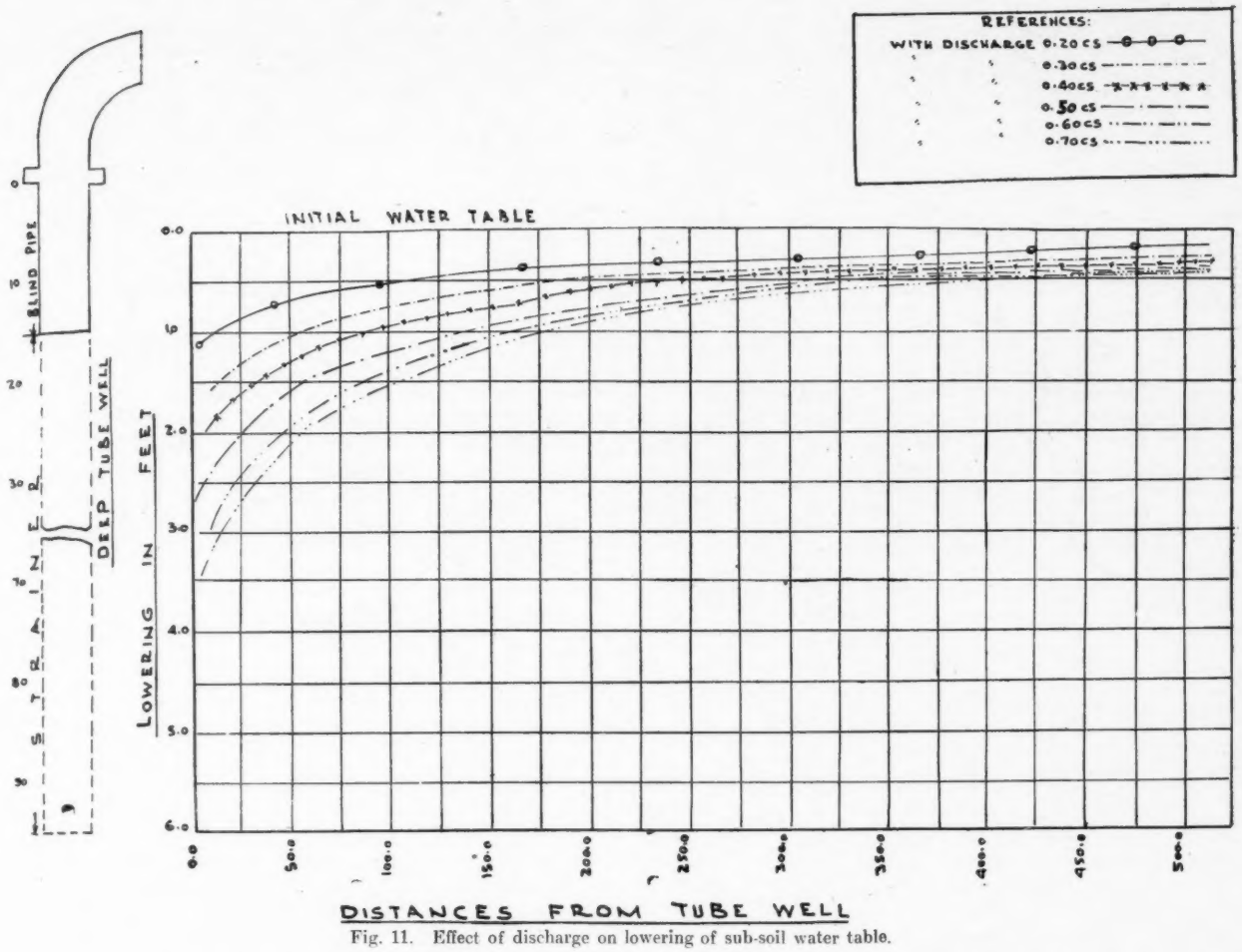

Fig. 11. Effect of discharge on lowering of sub-soil water table.

2. Rate and extent of lowering of water table with moderately deep tube well are smaller and lesser than with the shallow tube well.
3. Effect on lowering with tube wells on all sides is almost similar and gives similar pattern of draw down curves.
4. Zone of influence and ground water divide obtained with deep tube well is smaller than those obtained with shallow tube well.
5. Discharge of tube wells had a direct bearing on the lowering of water table, zone of influence and ground water divide. These were greater with greater discharges.
6. Lowering of water table caused by shallow tube well of 24 ft. filter length or 40 ft. filter length was almost the same for the same discharge.
7. Tube wells of greater filter lengths gave greater discharges. The maximum discharges obtained with filter length of 24.16 ft, 40.0 ft and 80.0 ft were 0.49 cs, 0.60 cs and 0.79 cs respectively.

Recommendation

Shallow tube wells not only give a reasonable discharge for irrigation purposes but also cause quick and effective lowering of the water table. It is recommended that this type of tube wells i.e. with 35 ft to 40 ft filter and about 12.0 ft to 15.0 ft blind pipe below sub-soil water level, may be adopted provided the quality of ground water available and the strata formation conditions permit. This will help to maintain balance between withdrawals and additions to the underground reservoir and prevent water-logging.

Table I

PROFILE NOTES OF BORE HOLES

S.No.	Depth of bore holes in feet	Colour of the soil	Nature of soil
1.	0-4	Brown	Hard clay
2.	4-6	,,	Hard clay with small kanker
3.	6-7	Dark brown	Fine sand, clay and small
4.	7-10	,,	kanker fine sand, clay and big kanker
5.	10-13	,,	Fine sand, clay and small kanker
6.	13-17	Blackish white	Medium sand and kanker
7.	17-30	,,	,, ,, ,, at 29-30 ft.
8.	30-31	Black	Fine sand
9.	31-36	,,	,, ,, and clay
10.	36-37	Brown	Medium sand
11.	37-64	Blackish clay	,, ,, and kanker at 39-41, 43-47, 50-51, 52-53 •55-58 ft.
12.	64-80	,, white	Medium sand and kanker at 66-68, 69-70, 71-72
13.	80-81	Black	Clay
14.	81-82	Blackish white	Medium sand & small kanker
15.	82-84	,,	,, ,. and clay
16.	84-87	,,	,, ,, ,, kanker at 85-86 ft.
17.	87-90	,,	Medium sand, clay and kanker at 88-90
18.	90-94	,,	Clay, Coarse sand and kanker
19.	94-103		Coarse sand and Kanker
20.	103-106	Blackish white	Coarse Sand & Clay clods and
21.	106-109	Black	Kanker Clay, Coarse Sand & Kanker at 108-109
22.	109-117	Blackish white	Coarse Sand and Stones
23.	117-118	,,	Coarse Sand and Clay
24.	118-123	,,	Coarse Sand
25.	123-124	,,	Coarse Sand and Clay clods
26.	124-125	,,	Coarse sand with kanker.

Table II

Analysis of Soil Samples from Bore Holes

S.No.	Depth in fect	P.H.	T.S. %	% clay	% Fine silt	% coarse silt	% sand	% kanker
1.	0-1	7.90	.40	20.15	10.65	7.20	63.00	Nil
2.	1 2	8.0	.04	20.30	7.70	6.00	66.93	,,
3.	2-3	8.05	.73	22.85	13.05	11.90	53.34	10.8
4.	3-4	8.0	.77	13.90	5.10	6.65	75.177	8.62
5.	4-5	8.35	.65	12.40	4.95	6.65	76.691	Nil
6.	5-6	8.50	.23	11.10	6.55	6.35	76.748	4.28
7.	6-7	8.10	.91	7.00	0.50	4.00	87.843	3.53
8.	7-8	8.65	.15	5.3	.70	2.15	93.008	Nil
9.	8-9	8.48	.25	3.50	2.25	1.80	92.46	,,
10.	9-10	8.60	.115				98.486	,,
11.	10-11	8.40	.30				99.062	,,
12.	11-12	8.20	.35				96.34	,,
13.	12-13	8.40	.22				96.203	,,
14.	13-14	8.40	.15				97.744	,,
15.	14-15	8.35	.21				97.173	,,
16.	15-16	8.35	.42				94.833	,,
17.	16-17	8.72	.11				98.366	,,
18.	17-18	8.45	.15				96 91	,,
19.	18-19	8.52	.095				94.520	3.26
20.	19-20	8.52	.21				99.560	1.72
21.	20-21	8.60	.16				99.480	Nil
22.	21-22	8.50	.15				98.182	,,
23.	22 23	8.50	.125				96.013	,,
24.	23-24	8.42	.25				98.315	,,
25.	24 25	8.48	.18				97.670	,,
26.	25-26	8.43	.20				99.140	,,
27.	26-27	8.65	.11				99.400	,,
28.	27-28	8.90	.07				98.813	,,
29.	28-29	8.90	.07				99 640	,,
30.	29-30	8.97	.08				99.436	,,
31.	30-31	8.67	.18				99.030	,,
32.	31-32	8.90	.11				99.272	,,
33.	32-33	8.85	.07				98.933	,,
34.	33-34	8.48	.17				98.494	,,
35.	34-35	8.48	.40	12.80	14.60	3.60	69.435	,,
36.	35-36	8.40	.21	4.00	0.50	1.00	95.75	,,
37.	36-37	8.62	.17	3.00	0.25	0.95	95.650	,,
38.	37-38	9.00	.11				98.76	,,
39.	38-39	8-5	.33				96.506	,,
40.	39-40	8.45	.36				99.072	,,
41.	40-41	8.78	.12				99.046	,,
42.	41-42	8.50	35				99.080	,,
43.	42-43	8.70	.17				98.708	,,
44.	43-44	8.62	.27				98.905	,,
45.	44-45	8.95	.095				99.19	,,
46.	45-46	8.40	.37	9.10	8.75	3.45	80.44	27.71
47.	46-47	8.40	.86	14.00	16.40	2.60	66.25	26.76
48.	47-48	8.88	.18				96.35	Nil
49.	48-49	8.35	.27				98.51	
50.	49-50	8.65	.18				99.69	,,

Table III

DISCHARGE OF TUBE WELL AND SUB-SOIL WATER LEVEL IN PIEZOMETER AT DIFFERENT INTERVALS.

S.No.	Time	Discharges.	Distances from Tube Wells															
			5.0 ft.		15.0 ft.		40.0 ft.		90.0 ft		310.0 ft		410.0 ft		460.0 ft.		495.0 ft.	
			Depth below N.S. (Feet)	Drop in Feet	Depth below N.S. (Feet)	Drop in Feet	Depth below N.S. (Feet)	Drop in Feet	Depth below N.S. (Feet)	Drop in Feet	Depth below N.S. (Feet)	Drop in Feet	Depth below N.S. (Fcet)	Drop in Feet	Depth below N.S. (Feet)	Drop in Feet	Depth below N.S. (Feet)	Drop in Feet
1	Before running		5.88	0.0	5.88	0.0	5.96	0.0	6.12	0.0	6.68	0.0	7.12	0.0	7.31	0.0	7.62	0.0
2.	After 5.0 min running	0.5395 cs	9.08	3.20	7.98	2.10	7.16	1.20	6.39	0.27	6.70	0.02	7.14	0.02	7.31	0.0	7.62	0.0
3.	,, 15.0 ,,	0.5342 cs	9.83	3.95	8.73	2.85	7.31	1.35	6.67	0.55	6.72	0.04	7.14	0.02	7.31	0.0	7.62	0.0
4,	,, 1.0 hrs ,,	0.5128 cs	10.27	4.39	9.02	3.14	7.66	1.70	6.96	0.84	6.88	0.20	7.25	0.13	7.41	0.10	7.66	0.04
5.	,, 4.0 hrs ,,	0.480 cs	10.81	4.93	9.50	3.62	8.07	2.06	7.16	1.04	7.02	0.34	7.41	0.29	7.58	0.27	7.77	0.15
6.	,, 8.0 hrs ,,	0.4618 cs	11.00	5.12	9.66	3.78	8.25	2.29	7.27	1.15	7.12	0.44	7.44	0.32	7.60	0.29	7.89	0.27
7.	,, 16.0 hrs ,,	0.4618 cs	11.14	5.26	9.82	3.94.	8.39	2.43	7.45	1.33	7.25	0.57	7.58	0.46	7.70	0.39	8.00	0.38
8.	,, 24.0 hrs ,,	0.480 cs	11.16	5.28	9.79	3.91	8.33	2.37	7.35	1.23	7.16	0.48	7.50	0.38	7.62	0.31	7.92	0.30
9.	,, 32.0 hrs ,,	0.480 cs	11.00	5.12	9.58	3.70	8.04	2.08	7.10	0.98	6.94	0.26	7.29	0.17	7.41	0.10	7.22	0.10
10.	,, 40.0 hrs ,,	0.480 cs	11.02	5.14	9.64	3.76	8.12	2.16	7.16	1.04	6.96	0.28	7.29	0.17	7.44	0.13	7.72	0.10
11.	,, 48.0 hrs ,,	0.480 cs	11.10	5.22	9.66	3.78	8.16	2.20	7.20	1 08	7.00	0.32	7.33	0.21	7.55	0.19	7.75	0.13

N.B. Slight Rise in Water table observed between 32 hrs and 40 hrs was due to rain fall.

Table IV

LOWERING OF WATER TABLE WITH DIFFERENT DISCHARGES AT DIFFERENT PERIODS WITH SHALLOW TUBE WELL.

S.No.	Distances from tube wells.	Time														
		5.0 min after running.			30.0 min after running.			1.0 hrs after runnning.			4.0 hrs after running.			8.0 hrs after running.		
		Discharges			Discharges			Discharges			Discharges			Discharges		
		0.3 cs	0.4 cs	0.5 cs	0.3 cs	0.4 cs	0.5 cs	0.3 cs	0.4 cs	0.5 cs	0.3 cs	0.4 cs	0.5 cs	0.3 cs	0.4 cs	0.5 cs
1.	5.0 ft.	2.21	3.20	3.20	2.67	33.4	4.21	2.77	3.54	4.39	3.13	4.09	4.93	3.27	4.29	5.12
2.	15.0 ft.	1.31	1.90	2.10	1.90	2.46	2.75	2.02	2.52	3.14	2.29	3.00	3.62	2.44	3.21	3.78
3.	40.0 ft.	1.02	0.87	1.20	1.05	1.27	1.33	1.15	1.39	1.70	1.40	1.75	2.06	1.50	1.93	2.29
4.	90.0 ft.	0.35	0.29	0.29	0.48	0.50	0.54	0.54	0.61	0.84	0.73	0.90	1.04	0.36	1.02	1.15
5.	310.0 ft.	0.04	0.02	0.02	0.08	0.04	0.04	0.10	0.10	0.20	0.22	0.25	0.34	0.27	0.33	0.44
6.	410.0 ft	0.04	0.02	0.02	0.07	0.04	0.04	0.07	0.07	0.13	0.15	0.17	0.29	0.21	0.25	0.32
7.	460.0 ft	0.02	—		0.02	—	0.04	0.02	0.02	0.10	0.11	0.13	0.27	0.17	0.19	0.29
8.	495.0 ft	0.00	—		0.0	—	0.02	0.0	0.02	0.04	0.04	0.10	0.15	0.10	0.18	0.27

Table V

LOWERING OF WATER TABLE WITH MODERATELY DEEP TUBE WELL (PRIVATE)

S.N.	Time	Distances from the Tube Wells							
		5.0 ft		10.0 ft		50.0 ft		100.0 ft	
		Water table below N.S. feet	Drop in feet	Water table below N.S. feet	Drop in feet	Water table below .N.S. feet	Drop in feet	Water table below N.S. feet	Drop in feet
	Initial reading	10.58	0.0	10.33	0.0	9.42	0.0	9.33	0.0
1.	After 5.0 min runniug	10.62	0.04	10.37	0.04	9.44	0,02	9.35	0.02
2.	,, 1.0 hrs ,,	10.87	0.29	10.75	0.42	9.75	0.33	9.54	0.21
3.	,, 4.0 hrs ,,	11.43	0.85	11.15	0.82	10.04	0.62	9.73	0.40
4.	,, 8.0 hrs ,,	11.48	0.90	11.14	0.81	10.08	0.66	9.77	0.44
5.	,, 10.0 hrs ,,	11.50	0.92	11.20	0.87	10.19	0.77	9.86	0.53

Table VI

LOWERING OF WATER TABLE WITH DISCHARGE OF 0.76 CS MODERATELY DEEP TUBE WELL.

Length of strainer = 80.0 ft.
Length of Blind pipe = 15.0 ft.

S.No.	Time	Distances from Tube Well													
		5.0 ft		15.0 ft		40.0 ft		190.0 ft		240.0 ft		265.0 ft		285.0 ft	
		Depth below N.S. feet	Drop in feet	Depth below N.S. feet	Drop in feet	Depth below N.S. feet	Drop in feet	Depth below N.S. feet	Drop in feet	Depth below N.S. feet	Drop in feet	Depth below N.S. feet	Drop in feet	Depth below N.S. feet	Drop in feet
1.	Initial reading	6.42	0.0	5.77	0.0	5.83	0.0	6.10	0.0	5.77	0.0	6.50	0.0	6.92	0.0
2.	After 5 min of running	8.65	2.23	7.96	2.19	7.79	1.96	6.25	0.15	5.92	0.15	6.63	0.13	7.06	0.14
3.	,, 15.0 min ,,	9.44	3.02	8.50	2.73	7.79	1.96	6.52	0.42	6.19	0.42	6.79	0,29	7.19	0.27
4.	,, 1.0 hes ,,	9.63	3.21	8.71	2.94	8.00	2.17	6.73	0.63	6.25	0.48	6.92	0.42	7.31	0.39
5.	,, 8.0 hrs ,,	10.21	3.79	9.25	3.48	8.52	2.69	7.33	1.23	6.75	0.98	7.40	0.90	7.81	0.89
6.	,, 24.0 hrs ,,	10.21	3.79	9.25	3.48	8.52	2.69	7.33	1.23	6.79	1.02	7.46	0.96	7.81	0.89
7.	,, 32.0 hrs ,,	—	—	9.29	3.52	8.60	2.77	7.38	1.28	6,83	1.06	7.46	0.96	7.85	0.93
8.	,, 48.0 hrs ,,	10.40	3.98	9.29	3.52	8.60	2.77	7.42	1,32	6.85	1.08	7.48	0.98	7.88	0,96
6.	,, 52.0 hrs ,,	10.65	4.23	9.40	3.63	8.65	2.82	7.44	1 34	6.92	1.15	7.56	1.06	7.92	1.00

	320.0 ft		370.0 ft		595.0 ft		690.0 ft		740.0 ft		775.0 ft		1030.0 ft		1240.0 ft	
1.	6.83	0.0	7.19	0.0	7.79	0.0	8.25	0.0	8.42	0.0	8.75	0.0	9.10	0.0	9.27	0.0
2.	7.06	0.23	7.29	0.10	7.85	0.06	8.27	0.02	8.44	0.02	8.75	0.0	9.10	0.0	9.37	0.0
3.	7.17	0.34	7.35	0.16	7.87	0.08	8.29	0.04	8.46	0.04	8.77	0.02	9.10	0.0	9.27	0.0
4.	7.29	0.46	7.48	0.29	7.94	0.15	8.33	0.08	8.52	0.10	8.79	0.04	9.10	0.0	9.27	0.0
5.	7.77	0.94	7.83	0.64	8.15	0.36	8.54	0.29	8.67	0.25	8.88	0.13	9.17	0.07	9.27	0.0
6.	7.79	0.96	7.88	0.69	8.19	0.40	8.54	0.29	8.71	0.29	8.88	0.13	9.19	0.09	9.27	0.0
7.	7.81	0.98	7.92	0.73	8.23	0.44	8.56	0.31	8,65	0.23	8.92	0.17	9.21	0.11	9.29	0.02
8.	7.81	0.98	7 92	0.73	8.23	0.44	8.56	0.31	8.63	0.21	8.96	0.21	9.21	0.11	0.11	0.02
9.	7.90	1.07	7.98	0.79	8.33	0.54	8.68	0.43	8.73	0.31	9.08	0.33	9.25	0.15	9.31	0.04

Table VII

LOWERING OF WATER TABLE WITH DISCHARGE OF 0.56 CS MODERATELY DEEP TUBE WELL

Length of strainer = 80.0 ft.
Length of Blind pipe = 15.0 ft.

S.No.	Time	Distances from Tube Well													
		5.0 ft		15.0 ft		40.0 ft		90.0 ft		190.0 ft		240.0 ft		265.0 ft	
		Depth below N.S.	Drop in feet	Depth below N.S.	Drop in feet	Depth below N.S.	Drop in feet	Depth below N.S.	Drop in feet	Depth below N.S.	Drop in feet	Depth below N.S.	Drop in feet	Depth below N.S.	Drop in feet
1.	Initial Reading.	6.81	0.0	6.00	0.0	6.08	0.0	6.08	0.0	6.38	0.0	6.04	0.0	6.77	0.0
2.	After 5.0 min running	8.46	1.25	7.48	1.48	7.04	0.96	6.64	0.56	6.41	0.08	6.08	0.04	6.81	0.04
3.	„ 15.0 min „	8.75	1.94	7.68	1.68	7.23	1.15	6.80	0.72	6.54	0.16	6.17	0.13	6.87	0.10
4.	„ 1.0 hrs „	8.98	2.17	7.92	1.92	7.46	1.38	7.00	0.92	6.75	0.37	6.29	0.25	7.00	0.23
5.	„ 4.0 hrs „	9.25	2.47	8.17	2.17	7.70	1.62	7.23	1.15	7.00	0.62	6.56	0.52	7.21	0.44
6.	„ 8.0 hrs „	9.31	2.50	8.25	2.25	7.79	1.71	7.31	1.23	7.15	0.77	6.65	0.61	7.33	0.56

	295.0 ft		320.0 ft		370.0 ft		590.0 ft		690.0 ft		740.0 ft		775.0 ft		1030.0 ft		1240.0 ft	
1.	7.25	0.0	7.12	0.0	7.46	0.0	8.10	0.00	8.50	0.0	8.69	0.0	9.00	0.0	9.40	0.0	9.60	0.0
2.	7.27	0.02	7.15	0.03	7.48	0.02	8.10	0.0	8.50	0.0	8.69	0.0	9.00	0.0	9.40	0.0	9.60	0.0
3.	7.29	0.04	7.19	0.07	7.52	0.06	8.10	0.0	8.50	0.0	8.69	0.0	9.00	0.0	9.40	0.0	9.60	0.0
4.	7.37	0.12	7.27	0.15	7.56	0.10	8.10	0.0	8.50	0.0	8.69	0.0	9.00	0.0	9.40	0.0	9.60	0.0
5.	7.58	0.33	7.46	0.34	7.73	0.27	8.17	0.07	8.56	0.06	8.71	0.07	9.02	0.02	9.42	0.02	9.65	0.05
6.	7.73	0.48	7.63	0.51	7.88	0.42	8.33	0.23	8.69	0.19	8.88	0.19	9.17	0.17	9.48	0.08	9.70	0.10

Table VIII

COMPARATIVE STATEMENT OF LOWERING OF WATER TABLE

S.N.	Distances fron Tube well	Shallow Tube Well				Deep Tube Well			
		5.0 min after running	15.0 min after running	1.0 hr after running	8.0 hrs after running	5.0 min after running	15.0 min after running	1.0 hr after running	8.0 hrs after running
1.	5.0 ft	3.20 ft	3.95 ft	4.95 ft	5.12 ft	1.65 ft	1.95 ft	2.17 ft	2.50 ft
2.	15.0 ft	2.10 ,	2.85 ,,	3.25 ,,	3.85 ,,	1.47 „	1.68 ,,	1.92 ,,	2.25 ,,
3.	30.0 ft	1.20 ,,	1.78 ,,	2.20 ,,	2.82 ,,	1.15 ,	1.32 ,,	1.59 ,,	1.90 ,,
4.	50.0 ft	0.70 ,,	1.02 ,,	1.48 ,,	2.03 ,,	0.84 ,,	1.0 ,,	1.25 ,,	1.52 ,,
5.	70.0 ft	—	0.65 ,,	1.02 ,,	1.57 ,,	0.67 ,,	—	1.02 ,,	1.30 ,,

Table IX

LOWERING OF WATER TABLE WITH VERY SHALLOW AND MODERATELY DEEP TUBE WELLS WITH 0.5 CS DISCHARGE

S.No.	Distances from Tube well	Shallow Tube Well wath 24.0 ft filter length					Moderately Deep Tube Wells				
		5.0 min after running	30.0 min after runntng	1.0 hr after running	4.0 hrs after running	8.0 hrs after running	5.0 min after running	30.0 min after running	1.0 hr after running	4.0 hrs after running	8.0 hrs after running
1.	5.0 ft	3.00	3.62	3.87	4.41	5.62	1.75	2.00	2.15	2.40	2.48
2.	15.0 ft	1.88	2.40	2.73	3.13	4.0	1.48	1.75	1.88	2.16	2.25
3.	40.0 ft	0.67	1.12	1.27	1.67	2.22	0.94	1.19	1.33	1.58	1.64
4.	90.0 ft	0.21	0.50	0.63	0.88	1.25	0.57	0.72	0.88	1.07	1.23
5.	190.0 ft	0.08	0.25	0.33	0.50	0.80	0.12	0.25	0.39	0.64	0.75

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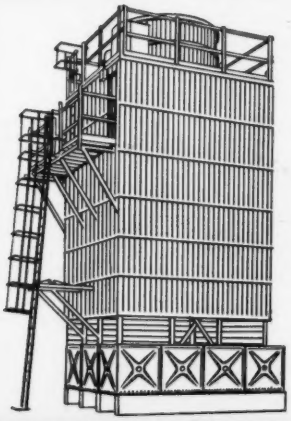

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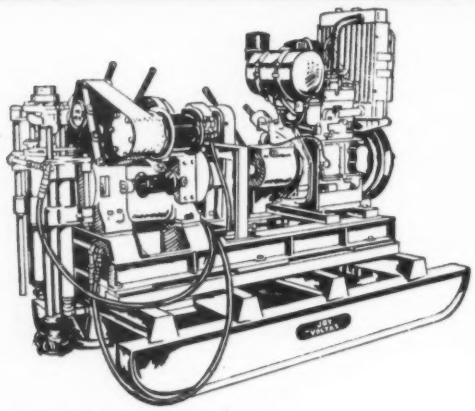

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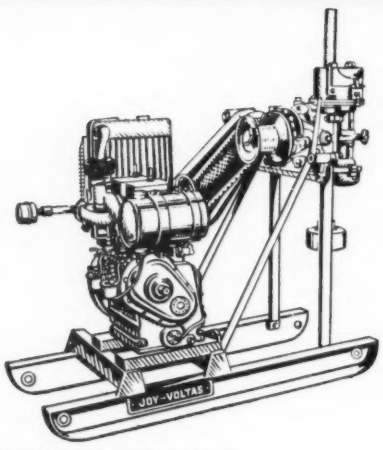

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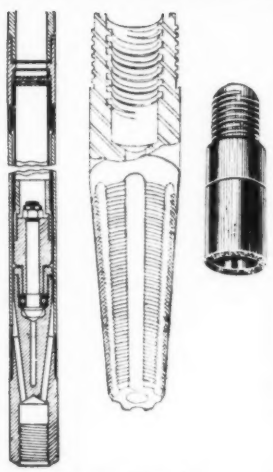

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Table X

LOWERING OF WATER TABLE WITH DIF FERENT DISCHARGE—DEEP TUBE WELL

S.No.	Distances in feet	Time: 5.0 min after running						30.0 min after running					
		0.2 cs	0.3 cs	0.4 cs	0.5 cs	0.6 cs	0.7 cs	0.2 cs	0.3 cs	0.4 cs	0.5 cs	0.6 cs	0.7 cs
1.	5.0	0.86	1.14	1.27	1.75	2.21	2.16	0.90	1.25	1.66	2.00	2,52	2.60
2.	15.0	0.75	0.94	1.17	1.48	1.84	1.98	0.79	1.11	1.48	1.75	2.23	2.35
3.	40.0	0.52	0.67	0.73	0.94	1.16	1.23	0.56	0.78	1.02	1.19	1.56	1.67
4.	90.0	0.32	0.41	0.41	0.57	0.65	0.70	0.36	0.52	0.64	0.72	0.98	1.06
5.	190.0	0.04	0.07	0.08	0.12	0.07	0.13	0.14	0.17	0.23	0.25	0.32	0.38
6.	240.0	0.02	0.04	0.02	0.07	0.06	0.06	0.04	0.14	0.14	0.19	0.25	0.27
7.	295.0	0.02	0.02	0.02	0.06	0.04	0.07	0.08	0.13	0.11	0.19	0.16	0.19
8.	370.0	0.02	0.03	0.02	0.06	0.04	0.05	0.04	0.08	0.13	0.15	0.12	0.15
9.	590.0	0.02	0.02	—	0.03	—	0.03	0.02	0.04	0.02	0.05	0.02	0.06
10.	690.0	—	—	—	0.02	0.02	0.02	—	0.06	0.02	0.06	0.04	0.04
11.	775.0	—	—	—	0.02	0.02	0.02	—	0.02	0.02	0.07	—	0.02
12.	1030.0	—	—	—	—	—	—	—	—	—	0.02	0.02	—

S.No.	Distances in feet	1.0 hr after running						2.0 hrs after running					
		0.2 cs	0.3 cs	0.4 cs	0.5 cs	0.6 cs	0.7 cs	0.2 cs	0.3 cs	0.4 cs	0.5 cs	0.6 cs	0.7 cs
1.	5.0	0.92	1.31	1.68	2.15	2.65	2.83	0.98	1.39	1.85	2.32	2.75	3.04
2.	15.0	0.79	1.19	1.48	1.88	2·38	2.56	0.85	1.27	1.62	2.08	2.44	2.73
3.	40.0	0.58	0.86	1.09	1.33	1.73	1.86	0.64	0.94	1.21	1.52	1.81	2.02
4.	90.0	0.38	0.60	0.70	0.88	1.12	1.25	0.46	0.66	0.81	1.02	1.23	1.39
5.	190.0	0.18	0.23	0.33	0,39	0.46	0.52	0.22	0.32	0.44	0.56	0.59	0.61
6.	240.0	0.13	0.19	0.23	0.23	0·35	0.39	0.19	0.27	0.33	0.34	0.46	0.52
7.	295.0	0.10	0.17	0.19	0.25	0·29	0.30	0.14	0.25	0.25	0.35	0.35	0.38
8.	370.0	0.04	0.13	0.15	0.19	018	0.21	0.12	0.17	0.19	0.27	0.27	0.30
9.	590.0	0.04	0.04	0.06	0.09	006	0.08	0.06	0.06	0.08	0.11	0.10	0.12
10.	690.0	—	0.06	0.02	0.06	0.04	0.04	—	0.08	0.04	0.06	0.06	0.08
11.	775.0	0.02	0.02	0.02	0.07	0.04	0.04	0.02	0.04	0.04	0.07	0.04	0.04
12.	1030.0	—	—	—	—	—	0.02	—	—	—	0.02	0.02	0.02

S.No.	Distances in feet	4.0 hrs after running						8.0 hrs after running					
		0.2 cs	0.3 es	0.4 cs	0.5 cs	0.6 cs	0.7 cs	0.2 cs	0.3 cs	0.4 cs	0.5 cs	0.6 cs	0.7 cs
1.	5.0	1.02	1.43	1.91	2.40	2·42	3.23	1.11	1.64	1.97	2.48	3.04	3.39
2.	15.0	0.91	1.31	1.71	2.16	2.63	2.94	0.95	1.48	1.77	2.25	2.73	3.02
3.	40.0	0.71	1.0	1.29	1.58	1.98	2.19	0.75	1,13	1.36	1.69	2.06	2.27
4.	90.0	0.50	0.70	0.91	1.07	1.37	1.52	0.54	0.83	0.98	1.23	1.41	1.62
5.	190.0	0.27	0.38	0.52	0.64	0·71	0.79	0.33	0,46	0.58	0.75	0.82	0.39
6.	240.0	0.23	0.33.	0.41	0.46	0.58	0.62	0.27	0.41	0.48	0.59	0.69	0.73
7.	295.0	0.19	0.32	0.34	0.40	0.45	0.48	0.25	0.38	0.40	0.50	0.54	0.57
8.	370.0	0.14	0.25	0.27	0.35	0·35	0.38	0.19	0.30	0.32	0.44	0.41	0.44
9.	590.0	0.06	0.10	0.10	0.17	0.12	0.16	0.11	0.17	0.16	0.23	0.14	0.20
10.	690.0	0.03	0.12	0.06	0.10	0.06	0.08	0.07	0.15	0.10	0.17	0.06	0.10
11.	775.0	0.04	0.06	0.06	0.09	0.09	0.06	0.06	0.08	0.08	0.15	0.05	0.08
12.	1030.0	—	0.02	—	—	0.04	0.04	0.02	0.02	0.02	0.02	0.06	—

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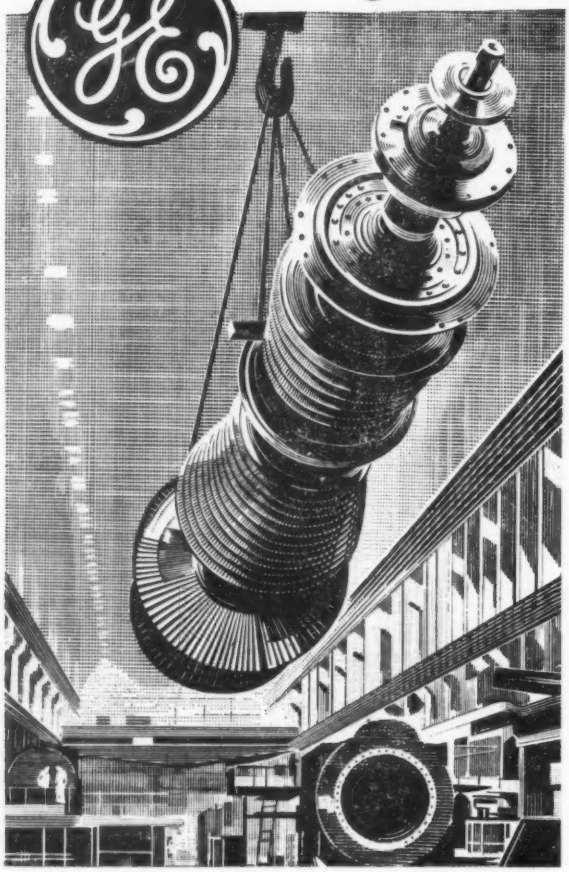

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AGI-2

Power Development in Austria

TO THE FOREIGNER IN AUSTRIA, FRESH PERHAPS from a visit to one of the concert halls in a city associated with Beethoven or Mozart, his first experience of one of Austria's great power-stations invariably comes as a surprise.

At Ybbs-Persenbeug, for instance, not far from the world-famous baroque monastery of Melk, a huge concrete dam over a quarter mile long, with 6 generators of a weight of 408 tons and a capacity of 45,000 kilowatts each, cuts the river Danube. Each of these generators, rotating noiselessly above Kaplan turbines, weighing nearly 1000 tons exerts a contrifugal force equal to the weight of two goods vans. And yet, there is absolutely no sinister feeling of monstrous power lurking behind the walls of the great bright halls where the human element is conspicuously absent, or in the long converging corridors lined with coloured mosaic that connect the various power-houses about 200 metres apart from each other. This combination of elegance and power testifies to the expertise and routine of Austrian technicians, who are among the world's most experienced designers of power-stations. It also is a feature of present-day Austria, which is not as widely appreciated as, for instance, her cultural activities.

Europe's Leading Exporter of Power

Austria with its 7 million inhabitants noted for their charm and hospitality, and possessing in the cities of Vienna and Salzburg two of Europe's foremost cultural centres, can to-day point with pride to 62 large power-stations, 50 of which are hydro-electric, in the Alps and in the Danube valley. Among the hydro-electric stations are giant affairs such as the two-stage Glockner-Kaprun plant in the Alps, or Ybbs-Persenbeug on the Danube, with a capacity of 33,000 and 162,000 kilowatts respectively. In 1959 the total output of Austria's power-stations was 14.75 billion kilowatt-hours, of which hydro-electric plants accounted for 10.94 billion, and thermo-dynamic plants for 3.81 billion. Austria exported one-sixth of this total, some 2.48 billion kilowatt-hours, to neighbouring countries, so keeping her leading position in Europa as an exporter of power.

The first power-stations were constructed as long as the beginning of the century, when factories for the processing of raw materials from the eastern provinces were set up in those parts of the Austro-Hungarian Monarchy (spread over a large part of Central and Eastern Europe) that correspond to present-day Austria, and in Bohemia and Moravia, subsequently incorporated in Czechoslovakia. The plants in this rapidly developing industrial area were fed by coal from Silesia and northern Bohemia, but in the mountainous western districts hydro-electric plants began to make their appearance, and this is the origin of the discrepancy between the power system of western Austria and that of the eastern part of what subsequently became the republic of Austria. Even to-day, with an integrated national network, there is still no connecting link between the two systems.

A Fresh Start with Power from the Alps

With the disintegration of the Austro-Hungarian Monarchy, and the emergence of Austria as a torso shorn of a great part of its industrial areas and sources of raw materials, the development of a new industrial potential with its own power-system became a matter of vital importance. The only available source of power in really abundant supply were the Alpine rivers, and this was the era of the great spurt in the construction in Austria of hydro-electric plants, and of the decline of thermo-dynamic plants, the coal for which had to be imported at considerable expenses from districts that were no longer part of Austria.

Despite considerable difficulties on the home front, Austria succeeded in constructing, up to the time of the world economic crisis, some 20 hydro-electric plants, the nucleus of the present system. Between 1918 and 1938 Austria's annual output of hydro-electric power rose from 895 million to 1,858 million kilowatt-hours, while over the same period thermo-dynamic output declined by one-half.

Amalgamation of the various Power Companies

The construction of power-stations and the distribution of power, hitherto in the hands of a number of different companies, were now taken over by newly-formed provincial companies which from time to time amalgamated various smaller firms.

One of the chief consumers of electric power were the Austrian Federal Railways, which pushed rapidly ahead with their programme of electrification started in 1914, and commissioned the construction of special "railway" power stations. Even in those days Austria was exporting electric power to western Europe, and the rapid development of Austrian production of electricity began to attract foreign investors as well as expert technicians from western

Europe who had an eye to the possibilities of harnessing Alpine rivers, and the Danube, for the production of electric power. The first design for the construction of the Ybbs-Persenbeug plant was actually submitted by the Swiss engineer Höhn as long ago as 1924.

Unification of the Network begins

The growing number of power-stations, and the increasingly complicated network of open-air-lines called for a complete reorganisation of the production and distribution of electric power throughout Austria. All the various systems in eastern Austria were co-ordinated by 1925, but a link between the newly-coordinated system in the east and the system operating in the western provinces, with its accent on export, was not considered practicable for the time being.

During World War II all Austria's power-stations and land-lines were absorbed into the German national network, so forming a component part of a single power-system operating from the great Hungarian plain to the Ruhr.

Reconstruction and new Methods

A mass of incomplete projects, and the total dislocation caused by four-power occupation were the heritage of the restored Austrian republic in 1945. It was of vital importance to get the national production of electric power on its feet again as soon as possible, and in order to use the available funds and labour as efficiently as possible, the production of electric power was nationalised during 1946 and 1947. A single holding company for all the various nationalised provincial companies was set up under a ministry : it was known as the "Osterreichische Elektrizitatswirtschafts AG," or "Verbundgeseilschaft" for short. A "Bundeslastverteiler" was also appointed, a central Federal authority responsible for the allocation of electric power throughout Austria.

Overall Planning of the Allocation of Billions of Kilowatt-hours

Austria being a federal republic, ownership of the nationalised provincial companies passed to the various provincial governments concerned. Though the influence of the central "Verbundgesellschaft" was primarily advisory, the existence of a supreme central authority controlling long-distance overland lines and transformers made it possible to coordinate the distribution of power throughout Austria, and to allocate where they were most needed the public funds raised at a national level for the development of the power-system.

Giant Power-Stations on a National Basis

One of the main tasks of the "Verbundgeselleschaft" is to finance the construction of large-scale power-station in the Alps or the Danube valley that are of national rather than local importance, and the cost of which is beyond the resources of any one single province. The construction of these giants is in the hands of the called "Sondergesellschaften" (special companies), 60% to 70% of their share capital being provided by the State from domestic or foreign loans or from foreign credits. The State's shares are administered by the "Verbundgesellschaft". The rest of the share capital of the "Sondergesellschaften" is put up by the provinces most closely concerned in the project. One example of a "Sondergesellschaft" is the "Osterreichische Denaukraftwerke AG" (DoKw), which aims to construct 13 large-scale power-plants on the Danube, with a capacity of 2.07 million kilowatts. At the time of writing, two of these projects have been completed, and work on a third has been started. Even the mighty Glockner-Kaprun project, unique in Europe, was financed by a "Sondergesellschaft", with the State holding the majority of the shares.

Proportion of Thermo-Dynamic to Hydro-electric : 1 : 3

The idea of forming a supreme central "Verbundgesellschaft" made it possible to determine the ratio of hydro-electric to thermodynamic plants throughout Austria as 3 : 1, and eventually to attempt by inegration to establish one single system of electric power for the whole of Austria. The plan evisages a system for eastern Austria consisting of three circles, two supplying Upper Austria, Salzburg, East Tyrol, Carinthia, and Styria, and a third circle, intersecting the other two, to cover Vienna.

The nervecentres of this system are the Glockner-Kaprun plant in the Alps and the Ybbs-Persenbeug plant on the Danube, as well as the oil and natural gas plant at Korneuburg, near Vienna. The culmination of this scheme of national integration is to be the construction of a link between the system described above and the system operating in Tyrol and Vorarlberg: at present the links are only in the planning stage. The Western Austrian system, with transmission lines to three neighbouring countries, is one of Austria's most important export grids.

The eastern Austrian system and the smaller Carinthian complex of power-stations further south also have five transmission lines for export to Western Germany, and one each to Czechoslovakia and Yugoslavia. The total length of 110 kV and 220 kV landlines in Austria amounts at present to 3469 kilometres.

The Marshall Plan and Loans

By 1947, the experts had their plans ready for the extension of the network of power-stations and the coordination of the distribution systems, and it was even possible to estimate, at this early stage, Austria's probable consumption of power for the next ten years. Unfortunately, no capital was available in those days for the realisation of such grandiose projects, so it was particularly fortunate

Mooser Dam on left and Drossen Dam on right.

that in 1947 Austria was apportioned a share of European Recovery Plan (ERP) funds. In ERP's peak year 1951, 76% of the funds was allocated to electrification projects. Up to the end of 1957 the "Verbundgesellschaft" had received 3.2 billion schillings from this source.

Between 1953 and 1959 the balance required for Austria's schemes of power development was raised by 22 public guaranteed loans, and brought up to a total of 10.535 million schillings by means of 27 foreign loans guaranteed by the State. Funds were also allocated from the National budget and from the local budgets of the individual provinces concerned.

For the 1960-1963 programme, which provides among other projects for the construction of more Danube power stations, 6.423 million schillings have recently been allocated. It is hoped that this will enable Austria to satisfy the ever-growing internal demand, which is increasing by about 6% annually, as well as to meet the brisk demand from abroad.

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Trends in Hydraulic and Structural Design of Aprons of Barrages on Permeable Soil Foundations—II

(*Continued from May, 1961 issue*)

(3) Method of Electrical Analogy :

IF TWO OR MORE UNRELATED AND ESSENTIALLY different physical phenomena are governed by the same differential equation or set of differential equations, we can study one of them by means of the other. The analogy helps us to adopt the advancements of the technique in other physical phenomenon to our field of interest. These methods of analogy often prove to be the most useful tools at the disposal of the design engineer.

Pavlovsky was the first to suggest an electro-hydrodynamical analogy for the solution of problems of percolation of water through sub-soils of the hydraulic structures. The fundamental basis of the analogy is the similarity of Ohm's law in electricity to Darcy's law of percolation of water through porous media.

(*a*) *Theory of Electro-hydrodynamical Analogy* :

A problem in any physical system is generally governed by the following equations : (a) Equations of equilibrium, (b) Equations of continuity, (c) Boundary conditions. Combining (a) and (b) in the case of percolation of water through porous media, (as shown in the previous sections) we obtain Laplace's equation,

$$\frac{\partial^2 \phi}{\partial x^2} + \frac{\partial^2 \phi}{\partial y^2} = 0$$

In the case of flow of electricity through conductors,, corresponding to (a), we have Ohm's law which states

$$I = \frac{E}{R} = K E \qquad \ldots (14)$$

where I=current, E=Voltage, R=Resistance of the conductor, and K=Conductivity

The above equation when expressed in differential form reduces to

$$I_x = -K \frac{\partial E}{\partial x}; \; I_y = -K \frac{\partial E}{\partial y} \qquad \ldots (15)$$

The equation of continuity can be shown as,

$$\frac{\partial I_x}{\partial x} + \frac{\partial I_y}{\partial y} = 0 \qquad \ldots (16)$$

Substituting equation (15) in the equation (16) of continuity, we obtain,

$$\frac{\partial^2 E}{\partial x^2} + \frac{\partial^2 E}{\partial y^2} = 0 \qquad \ldots (17)$$

Thus the flow of electricity through homogeneous conducting medium is also governed by Laplace's equation.

Therefore if the boundary conditions are made similar, the equipotentials must be identical in both cases, and hence the problems of percolation of water through sub-soils of hydraulic structures can be solved by experiments on electrical model.

For the simulation of the boundary conditions, the physical quantities should be visualised in terms of analogous physical quantities of the other system. The analogous quantities in the two systems are tabulated as below :—

Sub-soil flow	Flow of Electricity through conductors
Velocity of seepage flow	Intensity of current flow
Length of path of percolation	Resistance of the conductor.
Hydraulic Potential at the point expressed as a ratio of the total loss of head	E.M.F. or the potential at the point expressed as a ratio of the total drop of EMF in the system.

(*b*) *Simulation of Boundary Conditions and Conducting Medium* :

The first step in the electrical analogy—model is the simulation of the boundary conditions and conducting medium. The cross-sectional profile of a typical weir or barrage founded on permeable foundations is shown in Fig. 1.

The upstream face AB where seepage flow enters can be called 100% equipotential line and the downstream face LM where the seepage finds its exit can be called 0% per cent equipotential line.

These two equipotential surfaces are simulated by copper strips in the model. The boundaries B C D E F G H K L and A P Q M represent the innermost and outermost streamlines and they are simulated by non-conducting strips such as pyralin etc. The strips are secured in position by non-conducting adhesives as plusticine, etc.

In addition to the boundaries the resisting medium of the porous sub-soil will have to be simulated in the electrical analogy model. Both solid and liquid conducting media may be employed.

In the case of solids tin foils of thickness of about 0.005", thickness pasted on stiff non-conducting material with an insulating adhesive are used. With the solid conducting media, the simulation of impermeable barriers such as sheet piling is simple since all that is required is to cut out from the conduction sheet, figures geometrically similar to the barriers being studied. Whereas the electrical analogy model with solid conducting media restricted to two-dimensional flow problems electric analogy model with liquid conducting media is quite flexible for use even for certain problems of three-dimensional flow. Liquid conducting media are various electrolytes ; such as solutions of sodium chloride, ammonium chloride and copper sulphate.

(c) *Simulation of Stratification* :

Problems of percolation of water through stratified sub-soils of hydraulic structures can be easily solved by the method of electrical analogy. The method of simulating variable permeability consists in adjusting the depth of the conductor at the place suitably. The permeability co-efficient at the region above N O is five times less than that below. Since permeability is analogous to electrical conductivity, which varies inversely as the depth of the conducting medium the depth of the conducting medium above N O has to be made 1/5th or that below this line.

(d) *Determination of Potential Distribution* :

Wheatstone's bridge circuit is used for tracting the equipotential lines. This method consists in connecting the electrones A B and L M, representing 100 percent and 0 percent equipotential lines to a potential divider of resistance R. The sliding contact of the potential divider is connected to one terminal of a null indicating device and the other terminal is connected to a probing pencil. If x per cent equipotential line is to be drawn, the sliding contact is moved to x percent 'R' on the potentiometer. The probing-pencil is then traversed on the conductor, and various positions giving null points are noted. These points can be plotted on a graph paper.

The method of electrical analogy furnishes the potential distribution at all points along and interior to the given boundary. But for the design of the aprons of barrages and weirs on permeable foundations, knowledge of potential distribution along the profile of the barrage alone will suffice. The potential distribution-diagram enables determination of both uplift pressure and exit gradients.

(4) Method of Relaxation

The analytical solution to the Laplace's equation for complicated boundary conditions is not possible. Southwell's relaxation method proves to be the most powerful of all the numerical methods to the solution of the Laplace's equation.

The seepage field is covered with a square grid and for each node of this grid, the Laplace equation is expressed in the finite difference approximation form by the relaxation

$$\phi_1+\phi_2+\phi_3+\phi_4-4\,\phi_0=0$$

Thus, instead of solving one partial differential equation, we have to solve a set of simultaneous equations of finite difference. But when the number of interior points and therefore the number of simultaneous equations become great, the exact solution of the equations is too laborious. In the relaxation method, some arbitrary values of ϕ are assumed at the nodes of the grid and since these values are not likely to be correct, the amount by which they fail to satisfy the above relation is known as 'the Residual' (F_0) and for each node, it becomes,

$$F_0=\phi_1+\phi_2+\phi_3+\phi_4-4\phi_0$$

The residuals obtained at nodal points after substituting the arbitrarily assumed values of ϕ are liquidated by the use of the relaxation pattern. The final values ϕ obtained after liquidating the residuals

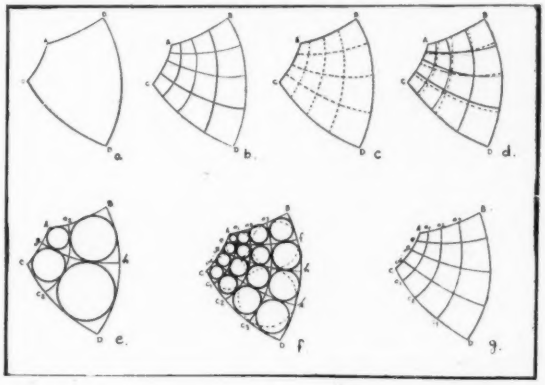

Fig. 4

indicate the pattern of potential distribution from which we can obtain the uplift pressures and exit-gradients.

(5) Graphical Methods

Methods of Forchheimer and Leliavsky : The graphical method of Forchheimer is based upon the criterion of "approximate squares". His method can best be explained with reference to Fig. (4A) which is a section of a weir built on a porous foundation. Keeping the sectional outline of the underneath of the structure in mind, one trial flow line (ac) adjacent to the boundary flow line should be drawn. The

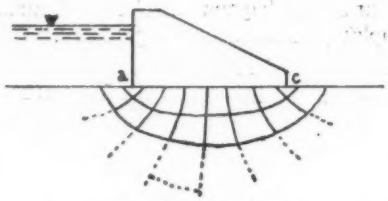

Fig. 4a

space in between the flow line (ac) thus drawn and the boundary flow line is then divided in such a way that the width of each section is approximately equal to its height. These sections can be described as approximate squares. Under each of these original squares a new square is built. The outer sides of these new squares will form the second flow line. This process is to be repeated until impermeable stratum is reached. The last flowline must coincide with the surface of the impermeable stratum. This serves as a check on the assumed alignment of the first flow line (ac). If it does not coincide, the above process is to be repeated with another assumed first flow line.

As the basic criterion of approximate squares is not rigid we cannot be sure whether the final solution obtained by Forchheimer's method is the exact one or not. This point can be explained with the help of Figures 4a to 4d. Fig. 4a represents the field to be investigated. AB and CD may be supposed to represent flow lines and AC and BD the equipotential lines. Satisfying the criterion of approximate squares an infinite variety of flow nets can be drawn in the field ABCD under investigation. Figures 4a and 4b represent two solutions which may be considered as equally good. It will however be apparent from a reference to Fig. 4d in which the two figures are superposed that the two flow nets did not coincide. Therefore both the flow nets might be considered unsatisfactory and moreover there is no independent standard which can be used as a common basis in comparing their advantages.

To improve Forchheimer's method, Leliavsky developed his criterion of circle on the basis of the principle that the four curves which intersect at right angles are all tangent to a common circle and therefore a perfect flow net should consist of fields in each of which it should be possible to place a circle.

In the same seepage field ABDC four co-tangent circles are placed and through the points of contact of the circles the flow line 'gh' and equipotential line 'a_2c_2' are drawn. Fig. 4f represents the process of obtaining a finer flow net while in Fig. 4g the circles are omitted and the flow net appears in its final form.

Notwithstanding the inherent defects of the graphical method it will be nevertheless realised that there is one and only one combination of sixteen ideal tangent circles which can form a regular pattern entirely filling the field enclosed by four boundary curves ABCD AC and BD. Therefore the circle method is the exact one.

EFFECT OF TAIL EROSION ON THE SAFETY CO-EFFICIENT AGAINST PIPING

Tail erosion is very frequent on barrages and weirs, built on alluvial river beds. In this sub-section it is intended to consider how tail erosion effects the safety co-efficient against piping and governs the required depth of the downstream sheet piling. Khosla laid down an empirical criterion to take into account tail erosion in the design of the understructure of the barrage but it was Leliavsky who approached the problem theoretically and substantiated the conclusions arrived at intuitively by Khosla.

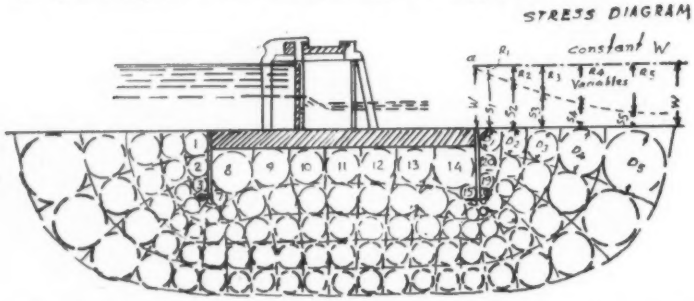

Fig. 5. Flow-net diagram prepared by circle method no scour (after Leliavsky).

Let us consider the flow-net diagram (Fig. 5) prepared by the circle method. The upward seepage forces S_1, S_2, S_3 etc. at the exit per unit volume of granular material are calculated according to the formula first suggested by Prof. Terzaghi

$$S = K\,Ws$$

in which, K=safety-co-efficient, W=specific weight, of water, S=exit-gradient, since there are 22 equipotential lines in the following diagram under consideration,

$$S_1 = \frac{K}{21}\,\frac{H}{D_1}$$

$$S_2 = \frac{K}{21}\,\frac{H}{D_2}$$

$$S_3 = \frac{K}{21}\,\frac{H}{D_3} \quad \text{etc.}$$

Taking downstream river bed level as the datum.

these forces are plotted upwards. The downward force, W, is then the submerged weight of the river bed soil. These two forces are found to balance each other at 'a'. It means that at the critical point, the submerged weight of the material is K times greater than the calculated seepage force. Since 'S' forces decrease at greater distances from the sheet pile, the resultant R of 'S' and 'W' is directed downwards and stable equilibrium therefore is unquestionable.

Now we shall examine the case of 'deep scour' as illustrated in Fig. (6). In this case, the resultants

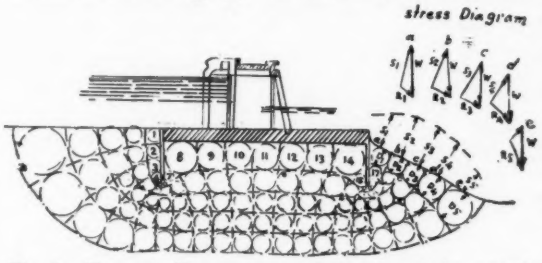

Fig. 6. Flow-net diagram prepared by the circle method deep scour (after Leliavsky).

at the points b and c are parallel to the line of slope and therefore the stability at *b* and *c* is by no means satisfactory.

So we may either increase the length of the apron to attain greater length of creep and thereby reduce the seepage forces or adopt a deeper downstream sheet-piling to drive the stream lines down into the deeper layers of the sub-soil.

The second solution is illustrated in Fig. (7) in which piling and scour-hole are assumed to be of

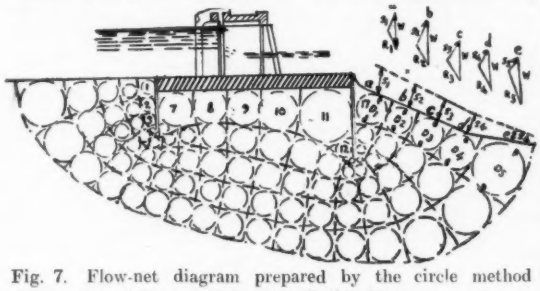

Fig. 7. Flow-net diagram prepared by the circle method deeper downstream cut-off (after Leliavsky).

equal depth in accordance with Khosla's suggestion. The slope of the resultant forces is now much steeper than that of the scour-holes and stability is consequently ensured.

It follows, therefore, that the methods adopted for reducing tail erosion and a deep downstream cutoff are both effective in preventing failure due to piping.

STRUCTURAL ANALYSIS AND DESIGN OF THE APRON OF A BARRAGE

In the following treatment, it is considered that the apron alone is the load-bearing element. Before attempting to make a detailed stress analysis, we have to decide whether to consider the apron as : (a) an elastic structure upon elastic foundation, (b) a rigid structure upon elastic foundation.

If the floor of the barrage is taken to be ideally rigid, the foundation pressure must be evenly distributed over its entire base area and the maximum pressure would consequently be less. Therefore, to be on the safe side, we must consider the apron of the barrage to be elastic.

The foundation pressure beneath the apron can be conceived as the sum of the two pressures. Uniformly distributed pressure f_1 which does not depend on the presence of the piers, and a variable pressure f_2 which is a function of the load transmitted by the piers. The total foundation pressure $f = f_1 + f_2$.

In the above equation f_1 depends on the weight of the apron, on the water load above it and the uplift pressure beneath the apron. At the toe of the pier the foundation pressure f_1 can be calculated by the formula

$$f_1 = t(\gamma - w) - 1/1\, h\, w,$$

where h is effective seepage head, t = thickness of the apron, γ = unit weight of the material of the apron, l & l_1 = are respectively the total length of the creep and the length of the creep downstream of the point considered.

To determine the pressure f_2 by the empirical method of Bligh and Rice-Oxley, it is assumed that the load is transmitted by the piers through the material of the apron at a slope of 2 to - (see Fig. 8a) and is therefore equally distributed on the soil over a width p + t under each pier while the rest of the apron is assumed to be altogether unaffected by the pier load. Within the limits of the width (p + t) the stress f_2 is then a constant and is given by

$$f_2 = \frac{p(f_o - w\,H_2)}{(p+t)} \quad \text{(see Fig. 8a)}$$

where f_o is the stress on the base of the pier calculated without taking uplift into account. (Fig. (8b) shows the distribution of the foundation pressure calculated in this manner. As an abrupt change of pressure intensities at two contiguous points of the same apron can only take place if the apron is already broken, the assumption on which the computation of diagram (8b) is based should be considered as a convention only. The foundation pressure distribution should be represented by a smooth continuous curve as the one shown in Fig. 8c, the theoretical basis of the computation of which is explained below.

Sub-grade Reaction Method as applied to the Design of the Apron of a Barrage :

The distribution of the foundation pressure over the bearing area of the apron of a barrage depends on both the physical properties of the supporting soil. foundation and on the elastic properties of the apron. The foundation pressure is a function of the deflection or deformation of the apron if the foundation material is perfectly elastic. The ratio between the

unit foundation pressure at a given point at the base of the apron and the deflection of that point is different for different points of the contact area. In the following analysis, it is assumed that the ratio between the unit foundation pressure and the settlement is

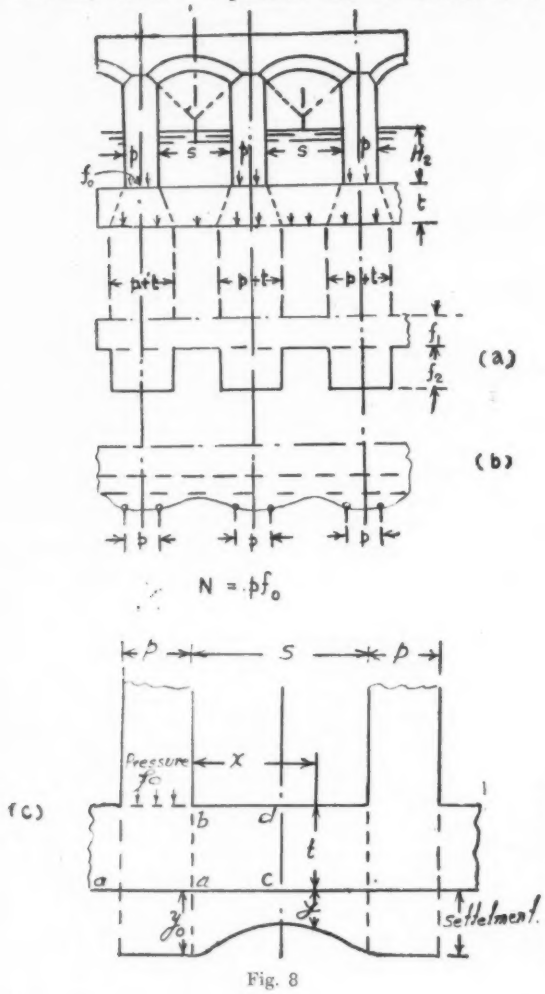

Fig. 8

the same for every point of the bearing area of the apron.

Let y be the deflection at a distance x from the origin which is taken at the section *ab*. (See fig. 8).

According to the assumption made above,

$$f_2 = K.y.$$

in which f_2 is the variable foundation pressure or sub-grade reaction and K is the co-efficient of sub-grade reaction.

The differential equation of the elastic curve of the apron on elastic soil foundation can be written as

$$EI\frac{d^4y}{dx^4} + Ky = f_o \qquad \ldots (18)$$

wherein E is Elastic Modulus of the material of the apron, I is Second Moment calculated as per unit, width of the apron i.e. '$t^3/12$', f_o is Stress at the base of the pier.

For the part of the apron, where $f_o = 0$, the solution of the equation (18) is,

$$y = \tfrac{1}{2}[\{A_1 e^{\psi} + A_2 e^{-\psi} \quad \text{Cos}\psi + \{A_3 e^{\psi} + A_4 e^{\psi}\} \sin\psi] \qquad \ldots (19)$$

$$\psi = \frac{X}{L}; \text{ and } L = 4\sqrt{\frac{4EI}{K}}$$

The slope of the elastic curve is,

$$\tan\theta = \frac{dy}{dx} = \frac{1}{L}\frac{dy}{d\psi}$$

$$\text{i.e.} = \frac{1}{2L}\Big[A_1 e^{\psi}(\text{Cos}\psi - \sin\psi) - A_2 e^{-\psi}(\cos\psi + \sin\psi) + A_3 e^{\psi}(\cos\psi + \sin\psi) + A_4 e^{-\psi}(\cos\psi - \sin\psi)\Big] \qquad \ldots (20)$$

The shear Q is

$$Q = -EI\frac{d^3y}{dx^3} = -\frac{KL}{4}\frac{d^3y}{d\psi^3}$$

$$= \frac{KL}{4}\Big[A_1 e^{\psi}(\cos\psi + \sin\psi) - A_2 e^{-\psi}(\cos\psi - \sin\psi) - A_3 e^{\psi}(\cos\psi - \sin\psi) - A_4 e^{\psi}(\cos\psi + \sin\psi)\Big] \qquad \ldots (21)$$

The above equations are general and we have to determine the constants of integration A_1, A_2, A_3 and A_4 from the boundary conditions covering the problem under consideration.

The boundary conditions are,

1. The slope of the elastic curve in the middle of the span is zero owing to the symmetry of forces and deflections. i.e. $\tan\theta\beta = 0$.

2. For the same reason, the shear at mid-span is zero i.e. $Q\beta = 0$.

3. If we assume that the second moment of the apron under the pier is infinite, it follows that the slope of the elastic curve within the limits "aa" fig. 8c is zero i.e. $\tan\theta\alpha = 0$.

4. It follows therefore that settlement and pressures beneath the pier are all uniformly distributed.

$$\text{i.e.} \qquad Q\alpha = \frac{K y_o\, p}{2} - \frac{N}{2}$$

where 'y_o' is the const. settlement under the pier and $N = p f_o$, the total force at the base of the pier per unit width of barrage.

Now applying the above boundary conditions to the general equations.

From equation (20) we get,

$$\tan \theta\alpha = A_1 - A_2 + A_3 + A_4 = 0 \qquad \ldots (22)$$

from equation (21) we get,

$$\omega_\alpha = \frac{KL}{4}(A_1 - A_2 - A_3 - A_4)$$

$$= \frac{K y_o \rho}{2} - \frac{N}{2}$$

$$= \frac{K \rho}{4}(A_1 + A_2) - \frac{N}{2} \qquad \ldots (23)$$

for according to equation (19)

$$y_o = \tfrac{1}{2}(A_1 + A_2).$$

From equations (20 & 21) we obtain,

$$\tan \theta\beta = A_1 e\beta (\cos \beta - \sin \beta) - A_2 e^{-\beta} (\cos \beta + \sin \beta) + A_3 e\beta (\cos \beta + \sin \beta) + A_4 e^{-\beta} (\cos \beta - \sin \beta) = 0 \qquad \ldots (24)$$

also,

$$Q\beta = A_1 e\beta (\cos \beta + \sin \beta) - A_2 e^{-\beta} (\cos \beta - \sin \beta) - A_3 e + \beta (\cos \beta - \sin \beta) - A_4 e^{-\beta} (\cos \beta + \sin \beta) = 0. \qquad \ldots (25)$$

in which, 'β' is the value of ψ in the section *cd*.

i.e. $$\beta = \frac{S}{2L}$$

$$= \frac{S}{2\sqrt[4]{\frac{4EI}{K}}}$$

Adding equations (24 & 25),

$$A_1 e\beta \cos \beta - A_2 e^{-\beta} \cos \beta + A_3 e\beta \sin \beta - A_4 e^{-\beta} \text{ in } \beta = 0$$

Dividing the above equation by $\theta e^{-\beta} \cos \beta$ we obtain,

$$A_1 e^{2\beta} - A_2 + A_3 \tan \beta e^{2\beta} - A_4 \tan \beta = 0. \qquad \ldots (26)$$

subtracting equation (25) from equation (24) we get,

$$A_1 e\beta \sin \beta + A_2 e^{-\beta} \sin \beta - A_3 e\beta \cos \beta - A_4 e^{-\beta} \cos \beta = 0.$$

Dividing the above equation by $e^{-\beta} \cos \beta$ we have,

$$A_1 e^{2\beta} \tan \beta + A_2 \tan \beta - A_3 e^{2} \beta - A_4 = 0. \qquad \ldots (27)$$

The four equations for determining A_1, A_2, A_3 and A_4 are

$$A_1 - A_2 + A_3 + A_4 = 0 \qquad \ldots (22)$$

$$(L - \rho)A_1 - (L + \rho)A_2 - LA_3 - LA_4 = \frac{-2N}{K} \qquad (23)$$

$$A_1 e^{2\beta} \tan \beta + A_2 \tan \beta - A_3 e^{2\beta} - A_4 = 0 \qquad \ldots (27)$$

$$A_1 e^{2\beta} - A_2 + A_3 e^{2\beta} \tan \beta \tan \beta - A_4 \tan \beta = 0. \qquad (26)$$

Solving the above four equations,

$$A_1 = -A_3 = \frac{2N}{K\lambda}$$

$$A_2 = A_4 = \frac{2Ne^{2\beta}}{K\lambda}$$

where $\lambda = \rho (e^{2\beta} + 1) + 2L (e^{2\beta} - 1)$

Substituting the above values in equation (19)

$$y = \frac{N}{K\lambda}[e\psi (\cos \psi - \sin \psi) + e^{2\beta - \psi} (\cos\psi + \sin \psi)] \qquad \ldots (28)$$

and since $f_2 = k y$

$$f_2 = \frac{N}{\lambda}[e\psi (\cos\psi - \sin\psi) + e^{-2\beta - \psi} (\cos\psi + \sin\psi)]$$

The maximum pressure under the pier can be found by equating $\psi = 0$

$$\therefore \quad (f_2) \text{ Max.} = \frac{N(e^{2\beta} + 1)}{\rho(e^{2\rho} + 1) + 2L (e^{2\beta} - 1)}$$

This shows that (f_2) max. can be calculated by assuming that the load of the pier is evenly distributed over a width $\rho + 2L\ (e^{2\beta} - 1)/(e^{2\beta} + 1)$ which corresponds to an angle of distribution

$$\tan^{-1} \frac{t}{L} \cdot \frac{e^{2\beta} - 1}{e^{2\rho} + 1}$$

It should be realised that although this methods is based on a rational solution, the imaginary lines delimiting the zone of influence of the pier load, have no physical significance except that the numerical value of the maximum pressure (f_2) obtained by elastic analysis will be the same as that calculated by assuming that the pier load is transmitted through the apron at a slope of

$$\tan^{-1} \frac{t}{L} \cdot \frac{e^{2\beta} + 1}{e^{2\rho} - 1}$$

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Homage to Harvey Slocum

THE sudden death of Harvey Slocum, the leading dam construction expert known throughout the world and consultant to the Bhakra Dam project, was indeed a great tragedy and loss to India and to the world of engineers. He was seventy-four and died in harness at Nangal. He not only did a great job; he set a great example to our engineers, young and old. He worked hard even at this late age and never spared himself. By sheer merit and will-power he rose to the top. Although he was the most highly paid man in our country, his simple style of living was remarkable and contrasted brilliantly against the lordly and lavish ways of some of our engineers. His sense of duty and devotion to his work can be gauged from the fact that although he was contracted to stay in India only for 4½ months in a year, he sometimes continued to overstay for 8 months and more. And yet another aspect of his personality needs to be mentioned before we close this reference to him. He was fearless and never cared to conceal his thoughts on the ground that they may not be palatable to "authority". He could sympathise with frank ignorance but hated sycophancy and incompetence in the garb of an intellectual pose. He may have been right or wrong on issues; but his frankness was luminous in the background of engineering diplomacy.

Such men are rare in any country. They leave monuments of their activity behind. But more than that, they leave an example of living—living every moment of one's life richly and fruitfully—to others. If he has succeeded by setting his own example (he did not believe in precepts) in inspiring even a few engineers and workmen in this country, his life will be genuinely commemorated by multiplication of the example. We pay our deep homage and respects to the memory of Harvey Slocum and convey our sincerest condolences to his bereaved family.

ALL INDIA ENGINEERING SERVICE

It was indeed gratifying to learn that after all the Indian Government had made up its mind to establish an all-India Engineering Service and the Chief Ministers have also consented to the proposal. The engineers have been clamouring for it for a long time but the need for it was based on the interests of the country as a whole. Two facts have underlined and might have facilitated the decision. As the experience accumulates with the execution of the Five-Year Plans (and each succeeding one grows staggering in size and complexity) the importance of engineers is being realised more clearly than ever before. On the other hand, the fissiparous tendencies in the form of regionalism, linguism, and communalism have so grown dangerously in the last few years that the concept of development of the *whole* country was fading in the background. The all-India character of engineering, medical and forest services, like the IAS and IPS and the Army will to some extent be able to check these undermining tendencies. We welcome the decision most heartily and let us hope bureaucratic procedures or regional petty mindedness will not come in the way of its quick execution.

THE "MANAGEMENT SECTOR"

Another major administrative bottleneck is holding back progress. This was again raised in a most forthright manner by Maj. General Harkirat Singh, President of the Institution of Engineers (India) at a Seminar on Economy and Efficiency of Engineering Enterprises held recently at Delhi, attended by over 400 leading engineers concerned. He said, if a high level of efficiency was to be achieved, it was essential that the management and administration was well-trained in the know-how of the enterprises they managed.

There could be no great technical development and progress, if technical men had to function within the iron cage of bureaucracy.

There was general agreement that economy did not mean curtailment of developmental activities. On the other hand, it should be associated with increased growth. Economy should compel a spirit of inquiry in governmental ways of thinking.

Describing the organizational methods, procedures, rules and regulations as old and outdated, the General said these regulations, and indeed the whole bureaucratic setup, were strangling the engineering enterprises.

"It is said there is no person more dangerous than a layman with pet ideas. That is what happens when laymen are put in authority to come between the democratic government of the country, who know what the public needs, and the technical men who know how to provide for the needs of the public".

The secretary of a Ministry, who had the power to overrule the technical head of the department,

had no responsibility for giving effect to technical or departmental proposals. "He only checks and criticizes. He does exercise influence of caution. But then too much caution does not achieve big results. Hence you have a pattern where there is authority without responsibility and responsibility without authority."

The General said it was a ludicrous situation where young officers of the civil service of a little more than 12 years of service trying to boss over experienced engineers of nearly 25 years service. In certain States they had even been given the authority to write confidential reports on chief engineers. It was no wonder, therefore, that engineers in the public service were frustrated and efficiency of engineering enterprises was low.

These are indeed very strong words. Perhaps more politely worded statements made earlier did not create any impression. We know from our personal contacts with senior engineers that they are very embittered over the matter. It is also true that quite minor officers on the civil side report on senior engineers and these carry weight. Apart from everything else, at this crucial phase of our economic development managerial skill is *the* most valuable thing. We suggest that with this bias in mind the problem be discussed at an informal level by the Indian Institute of Public Administration, and if necessary, the Institute prepare its own frank report after visiting some of the key projects in the country and circulate it in advance among the participants to the proposed discussion. Our impression is that managerial posts in many important engineering projects are recruited from among non-technical people and, as managerial merit is no criteria, nepotism has been rampant, of course not excluding political nepotism.

Professor Galbraith has been recently emphasizing the importance of the "management" sector in making the Third Plan a success. Occasionally we hear that people in authority have given a sympathetic ear to this view. But we seriously suspect if the matter has received the major importance it deserves. Under the Third Plan we have taken a major bite and we can not now vomit it out. The problem has to be faced. The material and capital resources we may secure but the whole question boils down to whether we can "manage" the whole business of execution. And here "delivering the goods" counts; not the question of this Service wing or that.

In this larger setting, Mr. Lal Bahadur Shastri's suggestion of setting up a Cabinet Committee is good but not enough. The problem is much larger. It needs the *major* and *continued* attention of the topmost leadership, of course with the advice of competent experts. Mr. Appleby has reported more than once. But has his underlying message been really taken to heart? We wonder.

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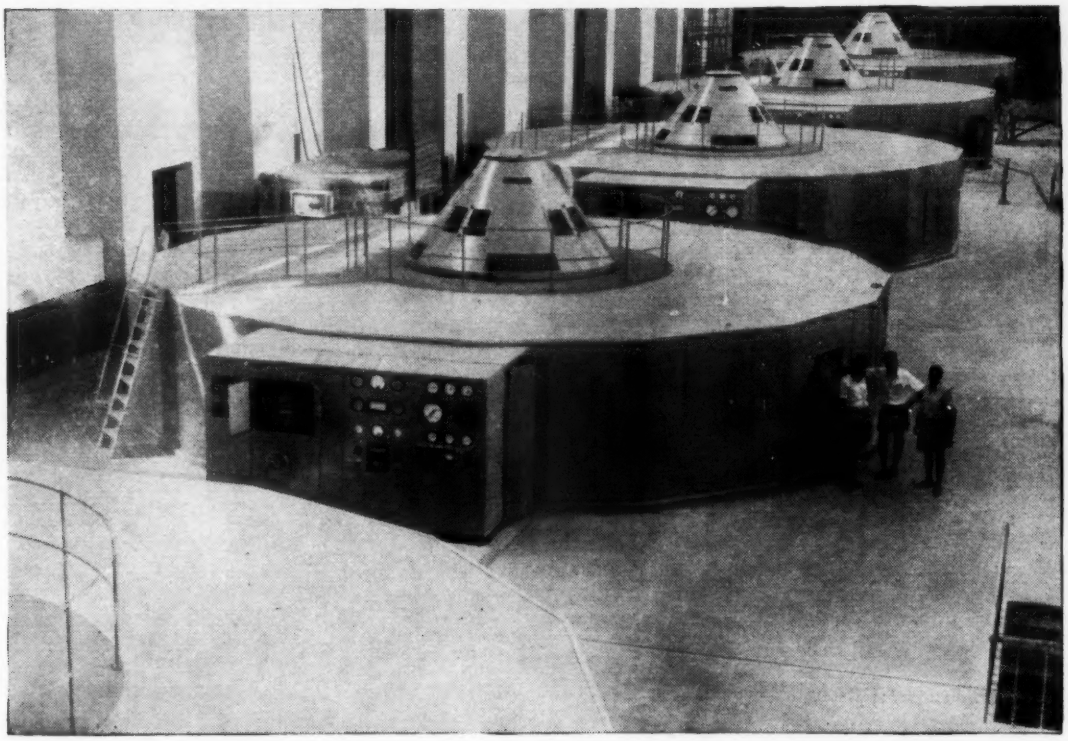

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Our Contributors

Mr. S. R. Vasudev: After graduating from the Government College, Lahore, passed the Diploma of Engineering from the Thomason Civil Engineering College, Roorkee in 1927 when he joined the Punjab Irrigation Department. After working in various capacities in the Punjab Irrigation Department where he was in charge of maintenance and construction of irrigation works, was deputed to the Central P.W.D. from 1942 to 1945 for construction of airfields during the War. On return to the Punjab Irrigation Department, was Under Secretary to the Government of Punjab in 1946 and was later transferred in the same capacity with the then East Punjab Government. In December 1948 was appointed as Superintending Engineer, Ferozepur Canals Circle, and in 1950 was deputed to the Government of India in the Central Water and Power Commission as Project Officer (Superintending Engineer), Rajasthan Canal Circle in which capacity organized surveys in the Rajasthan Canal Area and prepared the Preliminary Report on the Rajasthan Canal Project in 1953. Worked as Secretary, Tungabhadra Board, from 1953 to 1955 when appointed Superintending Engineer, Investigation Circle, in the Central Water and Power Commission. In 1955 took over as Director, Waterways Irrigation and Navigation, and worked in that capacity until retirement in June 1958. As Director WIN, contributed several articles and papers like development of inland navigation in India, Canal water-rates and Agricultural income, economical use of irrigation water etc. besides two papers (Soil Water Relationship and Irrigation and Drainage) for the International Irrigation and Drainage Commission in 1957. Since January 1961 working with the United Nations Special Fund after a brief interval of work with a business concern in Delhi.

Dr. H. L. Uppal passed his M.Sc., in 1928, started work as a Scholar of the Punjab University in Irrigation Research Laboratory. He worked on sub-soil flow and the design of works on the permeable foundations. In this field, he contributed a number of papers which exploded Bligh's Creep Theory and placed the design of works on sound foundations. He was awarded the degree of Doctor of Philosophy in Science on "Study in Physics of sub-soil water flow with special reference to design of works". He was appointed Head of the Hydraulic Laboratory in about 1940 and also Incharge of Malakpur Hydraulic Research Station. He also carried out work on river training and flood control. He was awarded a Medal of Punjab Engineering Congress and Kennedy' Gold Medal in 1945 for his paper on the "New Method of River Training". He was awarded the Fellowship of the National Institute of Sciences in 1950. In 1960, he was awarded the degree of Doctorate of Science on "Physics of Rivers and Design of Engineering Structures". He worked as Director, Irrigation and Power Research Institute from 1952—1960 (February) (Superintending Engineer's Grace). Since 1960 he has been working as Director, Irrigation and Power Research Institute and at present also Director, Land Reclamation in the rank of a Chief Engineer. He is also a Member of the Institute of Engineers, India.

Dr. Uppal has been to U.S.A., Canada, Middle East, Iran, Iraq and also to U.K. and Europe several times. He is a member of several Expert Committees both of the State as well as of the Government of India.

The Directorate of Land Reclamation and Irrigation and Power Research comprises of Hydraulics, Physics, Chemistry, Soil Mechanics, Ground Water Resources, Building Materials, Cement and Concrete, Land Reclamation, Mathematics and Statistics. Besides research work there is a lot of research-cum-extension work in the field being carried out by a number of Executive Engineers under his charge. There are several new apparatuses designed by him and new methods evolved by him.

During the tenure of his office in different capacities, he has published more than 130 papers on

different subjects in various National and International Engineering Organizations. Besides he has published books on:—

(i) Flood Control, Drainage and Reclamation in Kashmir Valley,
(ii) Sediment Control on Rivers and Canals, Vol. I & II,
(iii) Crop Pattern for Bhakra Canal Area,
(iv) Reclamation of Alkali Waste Land, etc.

Recently he evolved a new method for river diversion which has altogether revolutionised methods for training of rivers and flood control.

Mr. Gajindar Singh passed his B.Sc., (Honours) from the Punjab University in 1937. After passing his M.Sc., from the same university, he joined Irrigation and Power Research Institute, Punjab in 1938. But for a short spell of a year and a half for which he served on the India Air Forces, he remained on the staff of Irrigation and Power Research Institute Punjab throughout.

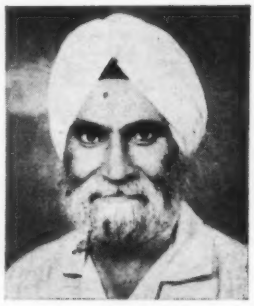
For 22 years of his service he worked in Hydraulic Section on different problems relating to Design of Hydraulic Structures such as Dams and its appurtenant works, Barrages, weirs, Anicuts, Power Canals, Spillway Syphon, Cross Drainage Syphon, Canal Falls, Sediment Excluders and silt ejectors etc. and on a large number of river models for the design of River training, flood control, and drainage works. As a part of this he examined the designs of different Levees, River embankments, Dykes, spurs and permeable screens.

His special subject of study has been the Design of Energy Dissipation Devices for High Head Hydraulic Structures. In the recent past, he evolved a new device known as Double Jet Energy Dissipator in which the sheet of water splits up into jets and strike against each other in the air and dissipate energy.

Besides he evolved a new apparatus known as Air-Silto-meter for quick and accurate analysis of silt and sand samples.

Since 1960, he has set up a Ground Water Resources Section where work is being carried out on Geophysical Survey of different tracts, periodical water table observations, characteristics and water yield of acquifers, study of quality of ground water etc. Besides, experiments are being carried out both in field and in the laboratories to evolve suitable design of surface drains, Infilteration galleries, tube wells, etc. to remove water logging conditions in the state.

Mr. Amrik Singh got Master of Science degree in Agriculture from the Punjab University in 1939.



He has been working in the Irrigation and Power Research Institute as Research Assistant, Assistant Land Reclamation Officer and Assistant Research Officer, since 1939. Has also worked as Tehsildar Rehabilitation for two years after partition of India.

Has carried out soil surveys of Lower Bari Doab Canal area (West Pakistant), Gang Canal area (Rajasthan), Chambal Valley (Madhya Pradesh) and Amritsar Pilot Scheme area.

Has carried out various investigations on the reclamation of waterlogged and saline soils and design of tube wells for anti-waterlogging.

Mr. M. V. Rao is Reader in Civil Engineering at the Government Engineering College, Jabalpur. The first article of this service appeared in our May 1961 issue.

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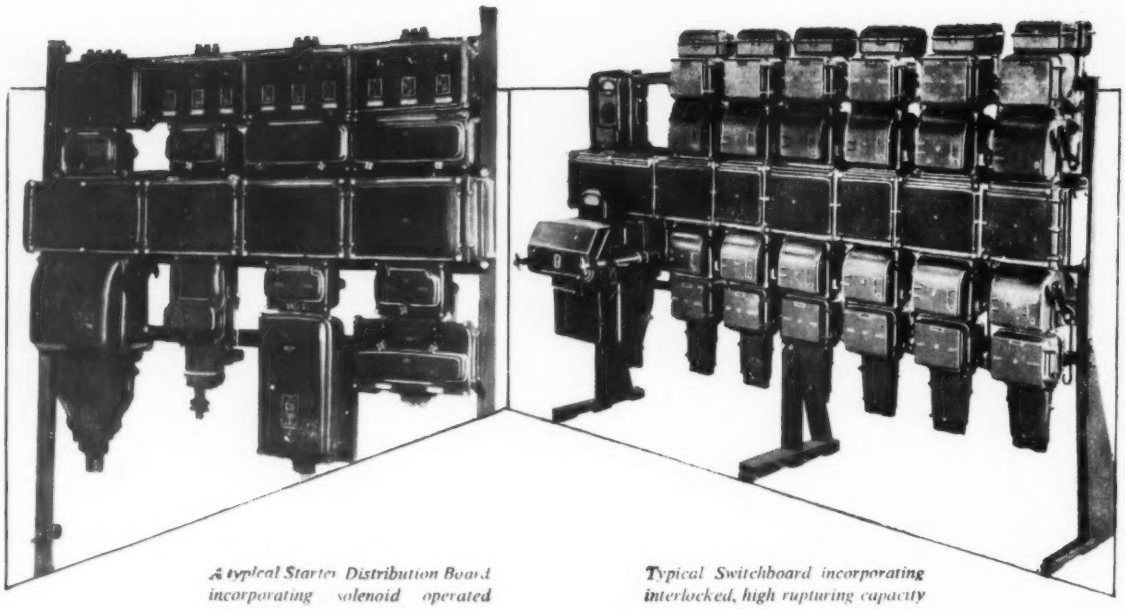

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